Alberta
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SUPPLY AND SERVICES

OLDMAN RIVER DAM

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SPILLWAY
DESIGN REPORT

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SPILLWAY
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Prepared for
ALBERTA PUBLIC WORKS, SUPPLY AND SERVICES

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1.0 INTRODUCTION

This report summarizes the design concepts, criteria and parameters which formed the basis for the design of the Spillway of the Oldman River Dam project. The Project Site is shown on Plate 1.

The Spillway will be located on the right bank of the Oldman River and in close proximity to the Dam as shown in Plate 2. The right bank location was selected following an economic optimization study involving the location and configuration of the Dam, Diversion Tunnels and Spillway. The Chute and Flip Bucket orientation aligns the discharge in the direction of the river valley downstream.

The main items of work in the Spillway Contract will include:

- final excavation for the spillway headworks, chute and flip bucket
- supply and installation of rock support
- foundation preparation
- supply and installation of the drainage system
- construction of the reinforced concrete spillway structure approximately 340 m long and 95 to 40 m wide comprising the headworks, chute and flip bucket

- supply and installation of embedded items in primary concrete for the gates and stoplogs
- construction of the precast prestressed concrete bridge structure
- placement of backfill and riprap erosion protection
- supply and installation of electrical and mechanical systems for the Spillway and Drainage Tunnels
- supply and installation of instrumentation

This Design Report covers the works to be completed under the entire Spillway Contract. It also includes works that, for contracting purposes, have been included in other packages as follows:

ITEM OF WORK	CONTRACT
Excavation of approach channel and bulk excavation for the Spillway	Road Embankment No.1 and Dam
Gates and Stoplogs including secondary concrete	Gates and Stoplogs
Control System	Control System

1.1 GENERAL ARRANGEMENT

The general arrangement of the Spillway, as shown on Plate 3 has been developed using data from the Preliminary Engineering Report, hydraulic model studies, field investigations and optimization studies.

INTRODUCTION

Water will be conveyed from the reservoir to the headworks in an excavated approach channel. The 110 m wide headworks consist of concrete approach walls, two abutment monoliths, five ogee monoliths, and an ogee crest overflow controlled by seven 10 m wide vertical lift roller gates.

A two lane concrete bridge will be provided over the upstream end of the headworks with an access and maintenance walkway adjacent to the gate hoist structure. A rectangular reinforced concrete chute converges from an arc width of 97.6 m at the downstream end of the piers to a width of 40 m within a transition length of 240 m and at a grade of 6.4H:1V. A 40 m wide concrete flip bucket and 100 m long partially pre-excavated plunge pool will dissipate hydraulic energy.

2.0 HYDRAULICS

2.1 DISCHARGE CAPACITY

The 1:1000 year flood of 2970 m^3/s has been adopted as the Spillway Design Flood. The Spillway will pass this flood with the reservoir at the full supply level (FSL) at elevation 1118.6 m. During the Probable Maximum Flood (PMF), the inflow of 9320 m^3/s will cause the reservoir to surcharge to its maximum elevation 1124.5 m at the dam. The corresponding outflow will be 7200 m^3/s through the Spillway and 180 m^3/s through the low level outlets. Losses along the approach channel are estimated at 0.35 m.

If the low level outlets become blocked during the PMF, the reservoir would surcharge to elevation 1124.7 m and the peak outflow over the spillway would increase to 7300 m^3/s.

Table I presents the reservoir elevation - spillway discharge relationship for free crest discharge and partial gate openings.

2.2 HYDRAULIC MODEL STUDY

The hydraulics of the Spillway arrangement have been model tested in the laboratory and the laboratory report (NHCL (1987)) complements this report. The model study provided the basis for the final hydraulic design dimensions, assisted in the definition of the general arrangement and provided water levels and hydraulic forces required for design.

TABLE I
SPILLWAY DISCHARGE CAPACITY
Spillway Crest Elevation = 1110.0 m

RESERVOIR ELEVATION (m)	FREE CREST DISCHARGE (m^3/s)	PARTIAL GATE OPENING DISCHARGE (m^3/s) 0.5m	1.0m	2.0m	3.0m	4.0m	5.0m
1125.5	7832	418	880	1636	2519	3378	3973
1125.0	7498	410	862	1606	2465	3311	3907
1124.5	7165	402	844	1575	2411	3243	3840
1124.0	6833	394	825	1544	2356	3174	3772
1123.5	6503	386	806	1512	2301	3104	3703
1123.0	6175	378	787	1480	2246	3032	3633
1122.5	5852	369	768	1447	2190	2958	3562
1122.0	5533	360	748	1414	2134	2882	3488
1121.5	5218	351	728	1380	2077	2804	3411
1121.0	4907	342	708	1346	2019	2723	3331
1120.5	4601	333	688	1311	1961	2640	3246
1120.0	4298	323	667	1275	1902	2554	3155
1119.5	3997	313	646	1238	1842	2466	3058
1119.0	3698	303	625	1201	1781	2377	2958
1118.6 (FSL)	3455	294	608	1172	1732	2307	2875
1118.5	3400	293	603	1163	1719	2287	2854
1118.0	3105	282	581	1124	1657	2196	2744
1117.5	2814	271	559	1084	1594	2104	2629
1117.0	2527	259	536	1043	1531	2011	2509
1116.5	2247	247	513	1001	1467	1917	
1116.0	1972	234	489	958	1403	1821	
1115.5	1707	221	465	914	1338		
1115.0	1453	207	440	869	1272		
1114.5	1204	193	415	823			
1114.0	958	178	389	775			
1113.5	722	163	362				
1113.0	517	147	334				
1112.5	337	131	305				
1112.0	206	114					
1111.5	112	96					
1111.0	56						
1110.5	20						
1110.0	0						

2.3 SPILLWAY APPROACH CHANNEL

The excavated approach channel runs from the valley wall to the spillway headworks. The approach channel entry has a base width of 60 m, and invert elevation of 1099.5 m. This section remains constant until the beginning of the curved section at which point the channel sides diverge and the side slopes flatten until it reaches the headblock with a base width of 110 m. The channel invert remains constant at elevation 1099.5 m until it intersects the 5H:1V slope that extends 39 m upstream from the Spillway headblock. The channel has been designed to limit the velocities during the Spillway Design Flood to approximately 2 m/s and to limit the bend loss through the curved section.

2.4 HEADWORKS

The approach walls retain embankment fill in the area upstream of the gate structure and also form part of the transition section between the approach channel and the headworks. The headworks have an ogee crest overflow controlled by seven 10 m wide x 9.817 m high vertical lift roller gates in a radial converging arrangement in plan. The ogee crest is 8.6 m below FSL at elevation 1110.0 m. The upstream apron of the ogee will be at a maximum elevation of 1107.30. The upstream face is on a slope of 2H:3V. The profile at the nappe is based on a design head of 8.6 m using procedures described in the US Bureau of Reclamation "Design of Small Dams".

The pressure effects of wave action on the gate structure have been calculated in accordance with the US Army Corps of Engineers "Hydraulic Design Criteria". Maximum wave forces on the headworks are for a 1:1000 year wind with the reservoir at FSL. The total hydrostatic and wave pressure distribution is zero at elevation 1121.2 m and increases linearly to 11.96 m of head at elevation 1106.8 m. The pressure between the top of the gate (1119.817 m) and elevation 1121.2 m has not been included as the wave in this region would be spilling over the top of the gate and would not exert a horizontal force on the structure.

2.5 CHUTE

The Spillway chute converges from an arc width of 97.6 m at the downstream end of the piers to a width of 40 m within a transition length of 240 m. Beyond this point, the chute remains a constant width leading to a 40 m wide flip bucket. From headworks to the beginning of the flip bucket, the grade is a constant 6.4H:1V on chute centreline.

The chute walls are sufficiently high to contain the flows associated with the PMF with allowance for air bulking and waves. Flow depths and velocities along the chute have been determined from hydraulic gradient computations using Manning 'n' values of 0.016, 0.014 and 0.012 and have been measured in the spillway hydraulic model. Wall heights have been based on the model results for the PMF discharge. Wall heights are a constant 10.5 m to allow a minimum of 1.0 m of freeboard at PMF flow to accommodate air bulking and splash.

During passage of the Spillway Design Flood, maximum velocities computed using the lower 'n' value vary from 13.3 m/s at the headworks accelerating to 31.1 m/s at the flip bucket. These high velocities will require a fine tolerance to the surface finishes to minimize the potential for cavitation erosion of the concrete.

2.6 FLIP BUCKET

The energy will be dissipated by a flip bucket located approximately 100 m clear of the downstream toe of the Dam. The flip bucket consists of a 20 m radius vertical curve terminating at a flip angle of 40 degrees. The lip of the bucket is at elevation 1062 m which is about 1 m above the estimated tailwater level during the PMF.

During the Spillway Design Flood discharge, the impact area is between 80 m to 100 m downstream from the bucket lip. At lower discharges, it is closer to the structure and at higher discharges it is slightly further away.

Downstream of the flip bucket, a partially pre-excavated plunge pool will provide initial energy dissipation.

The impact of the plunging jet and the return currents thus induced are expected to cause extensive scour of the riverbed and erosion at the base of the valley walls. Measures to protect the toe of the Dam and the flip bucket base against undermining have been developed from the hydraulic model studies.

3.0 GEOTECHNICAL

3.1 GENERAL

In this section the geology, foundation conditions, and design parameters are discussed.

3.2 REGIONAL GEOLOGY

The regional geology has been previously described in the Preliminary Engineering Report (UMA/Acres (1986)) and in the Oldman River Dam Design Report (UMA/Acres (1988)). A summary is presented herein.

The general area in which the project is located comprises two physiographic regions, the Alberta Plain and the Rocky Mountain Foothills. The project site is situated on the Alberta Plain approximately 9 km northeast of the edge of the Rocky Mountain Foothills. The area is underlain by gently dipping Upper Cretaceous and Tertiary sedimentary rock units which form a broad, regional synclinal structure called the Alberta Syncline (Douglas, (1950) and (1970)).

The syncline comprises the Porcupine Hills formation and the underlying Willow Creek formation. The Porcupine Hills formation is a nonmarine clastic sequence consisting of interbedded sequences of cross-bedded, grey, buff-weathering, ledge-forming, calcareous sandstones with grey to dark grey siltstone and silty claystone mudrocks. The lower contact of the Porcupine Hills Formation is considered to be disconformable.

The underlying Willow Creek formation is a nonmarine, clastic sequence consisting of interbedded mudrocks and sandstones. The mudrocks consist of green, grey and purplish, claystones and siltstones. The sandstones consist mainly of poorly indurated, massively bedded and cross-bedded, fine grained, grey, quartz sandstone; some hard, thin bedded, fine grained layers with calcareous cement are also present. Mudrocks are generally more abundant than sandstones.

With the exception of some minor, local strata dislocations, no faults are known in the immediate vicinity of the project site; however, thrust faults are present at depth in the Willow Creek formation. These faults pre-date deposition of the Porcupine Hills formation because they do not cross the contact between the Willow Creek and Porcupine Hills formations (Bossart, (1957); Carrigy, 1971)).

The region has been repeatedly glaciated and hence the bedrock is mantled with overburden comprising glacial till, glaciofluvial and glaciolacustrine materials. Postglacial sediments include alluvial gravel and sand in terraces and floodplains along the river and stream valleys, colluvium on the valley slopes, and a discontinuous blanket of eolian silt on the upland areas.

3.3 SITE GEOLOGY

A detailed description of the site geology has been given in the Oldman River Dam Design Report (UMA/Acres (1988)). A brief description of the geology in the vicinity of the Spillway is given in the following section.

3.3.1 Overburden and Groundwater Level

The overburden soils on the Spillway site consist of topsoil, eolian silt, glaciolacustrine silt and clay, sand, gravel and clay till to a maximum depth of 16 m.

The existing groundwater level in the river banks is generally close to river level near the valley and rises within the range of elevations 1060 m to 1070 m in the area of the Spillway and the right abutment of the dam. Minor and probably localized perched water tables have been encountered.

3.3.2 Bedrock

The bedrock underlying the Spillway is part of the Porcupine Hills and Willow Creek formations. The Porcupine Hills formation overlies the Willow Creek formation and the contact is located at approximate elevation 1007 m at the Spillway headworks. Both formations consist of interbedded sequences of freshwater sandstone and mudrock (mudrock is defined as rock consisting of siltstone and claystone). It is believed that the rocks were deposited in a meandering fluvial system where sandy deposition was restricted to major channels. Deposition of fines occurred on levees and in flood basins.

Three major stratigraphic units comprise the Porcupine Hills formation and Willow Creek formation encountered during exploration at the project site. These have been designated the Lower Mudrock, Basal Sandstone and the Upper Mudrock sequences, for design purposes.

The Lower Mudrock Sequence underlies the Basal Sandstone Sequence throughout the site area at an approximate depth of about 45 m below the river level centreline. The Lower Mudrock Sequence comprises primarily interbedded claystone, silty claystone, clayey siltstone, and siltstone, and to a lesser extent, silty sandstone and sandstone. Considerable lateral and vertical variation exists within the strata; horizontal continuity of a given rock type at a particular stratigraphic level is, therefore, limited. Facies changes are frequent in the horizontal direction. The top of the Lower Mudrock Sequence is the uppermost bed of the Willow Creek Formation.

Thin, black, carbonaceous claystone bands generally up to 50 cm thick are found in the Lower Mudrock Sequence. The Lower Mudrock Sequence is considered to be a shallow river channel and floodplain type of deposit.

The Basal Sandstone Sequence extends from elevation 1007 m to 1045 m, and is divisible into three sections. The upper section, 11 m thick, and the lower section, 13 m thick, consist of uniform, very thickly bedded and very continuous strong sandstone. The central section, 14 m thick, consists primarily of sandstone, but contains several relatively thin subsections of interbedded siltstone and claystone. The thickness and continuity of these mudrock beds are highly variable. The Basal Sandstone is considered to be a deep channel type deposit. This Basal Sandstone is not actually exposed in valley bottom.

The Upper Mudrock Sequence overlies the Basal Sandstone Sequence and forms the foundation of Spillway and the abutments of the

Dam. This sequence comprises primarily claystone, silty claystone, clayey siltstone, and siltstone, but has considerable thicknesses of silty sandstone and sandstone.

Two unusually thick sandstone channel deposits occur in the Upper Mudrock Sequence. One occurs high in the left abutment of the Dam and is a maximum of 16 m thick. The second immediately overlies the Basal Sandstone downstream of the Dam and occurs in the flip bucket area. It crosses the present river valley and varies from 5 m thick in the left abutment to a maximum of 18 m in the right abutment; a thin basal layer of this channel sandstone forms the upper most bedrock in the downstream plunge pool area. These sandstone deposits are considered to be overbank and shallow channel type deposits.

The strata comprising the Upper Mudrock Sequence are similar to those in the Lower Mudrock Sequence in terms of horizontal and vertical variability and horizontal continuity. Similarly, thin, continuous, black, carbonaceous beds exist at regular intervals through the Upper Mudrock Sequence. These major carbonaceous beds are nearly site-wide in distribution and can serve as stratigraphic marker beds, while minor carbonaceous beds show lesser areal extent. Calcrete nodules also occur in zones in certain stratigraphic levels in the mudrock; these zones locally serve as marker beds.

3.4 FOUNDATION CONDITIONS

The bedrock of the Porcupine Hills formation comprises the foundation of the Spillway headworks, chute and flip bucket structures.

The headworks and the chute will be founded on unweathered, undisturbed rocks of the Upper Mudrock Sequence. These mudrocks contain subhorizontal structural discontinuities or seams which have lower shear strengths than the adjacent intact rocks. These seams are located at various elevations beneath the structures. Potential sliding surfaces will tend to pass along these seams.

The base of the flip bucket will be stepped at elevation 1035 m, 1043 m and 1048 m. The foundation material under the flip bucket consists of the Basal Sandstone Sequence from approximately elevation 1007 m to elevation 1045 m, and the channel sandstone from about elevation 1045 m to elevation 1055 m. Overlying the channel sandstone are predominantly mudrock of the Upper Mudrock Sequence. The channel sandstone unit includes at least three claystone and siltstone interbeds, approximately at elevations 1047 m, 1050 m and 1052 m. Laterally, the channel sandstone is limited and pinches out to the north and south along the river bank. Seams exists at approximately elevation 1036 m and 1030 m. These are continuous within the Basal Sandstone Sequence over the full lateral extent of the flip bucket foundation.

The channel sandstone is medium grained, strong, slightly weathered, thinly bedded (average thickness 200 mm) with medium sized blocks varying in volume between 0.08 to 0.2 m^3. Bedding is mainly horizontal with minor gently sloped trough cross-bedding. Jointing in this sandstone is both parallel and perpendicular to the bedding. Vertical joint sets strike 170° and 260°, and have joint spacings of an average 200 mm, and clean apertures 6-20 mm wide. From the drillhole data, on average there is one joint per metre.

3.5 BEDROCK SHEAR STRENGTH

3.5.1 General

A detailed assessment was made of the shear strength of the bedrock in the foundation of the headworks, chute, flip bucket and in the slopes of the spillway excavation. A comprehensive description of this assessment is given in the Oldman River Dam Design Report (UMA/Acres (1988)) and in Section 4.5.1. In this section a brief summary is given of the shear strength along those seams significant to the design of the Spillway structure.

3.5.2. Shear Strength Along Seams

The bedrock strength that will control stability is a combination of the strength across the beds and along the subhorizontal seams found to exist within the rock mass. The foundation investigations have concentrated on obtaining data relative to these seams. The strength, continuity, thickness, surface shape and orientation of the seams were analyzed using data from coring, drilling, laboratory testing and detailed mapping of the geology revealed in the test tunnel, test shafts, test trenches, and surface exposures, in bedrock exposures in the portal excavations for the Diversion Tunnels and in the construction access road in the river valley along the left river bank.

The strength of individual seams will be less than that of the adjacent intact rock due to tectonic deformation that has taken place, but will be somewhat greater than the minimum residual value measured in laboratory direct shear tests on samples taken from the seam. A design strength representative for each seam has been assessed by considering a "geological component" that recognizes the variability of the laboratory test data and of the site geology. The geological component takes into account the actual conditions in and along the seam, the varying rock types through which it passes, the filling material, which often contains fragments of clay, silt and rock, the shape and thickness of the seam, the dip and undulation of the seam, the asperities along the seam surface, the variations in residual shear strength on samples taken from the seam and that portion of peak shear strength that may still be available if the tectonic deformation that caused the seam was insufficient to reduce the shear strength completely to its residual value. The geological component assigned to each of these major seams also takes into account the size of the structure or sliding surface considered in the analysis. The shear strength that was assigned to each of the seams for use in design is a value considered to be representative of the actual in situ shear strength taking into account the laboratory test data and the geological component.

All of the seams that have been identified in the foundations do not have the same influence on the stability of the structures. Most seams can be disregarded due to their lack of continuity in the vicinity of the structure. The shear strength of fourteen seams that have the most significant effect on the design of the Spillway structures and of the excavated slopes are listed in Table II together with the effective angle of shearing resistance assigned for the design. The effective cohesion along all seams is zero.

The rationale for assigning representive shear strength values to the foundation rock is described in detail in the Oldman River Dam Design Report (UMA/Acres (1988)).

No shear strength tests were carried out on the material from the seams in the flip bucket foundation area during the design stage because of the difficulties experienced in obtaining core samples from drillholes that contained sufficient seam material for shear box testing and for a definitive evaluation of seam conditions. Sinking exploratory shafts to observe the seams in situ and to obtain large horizontal core samples for laboratory testing at the time would have been costly because of the thickness of soil and rock overlying the foundation elevation and because of the complications of access onto the river bank slopes. As a consequence interim values of the effective angles of shearing resistance for these seams were assigned using laboratory test data on samples from the foundation of the Dam.

A supplementary flip bucket investigation program to confirm the presence, continuity and shear strength of these seams, was carried out in May and June, 1989, as part of the Spillway contract after completion of bulk excavation for the structure by the Dam Contractor. The results of this investigation are presented in Section 4.5.1.

3.5.3 Bedding Plane Shear Strength

Between the continuous major design seams, an effective angle of shearing resistance of 17.5° along planes parallel to the bedding has been used for the design. The only exception is a significantly weaker zone between elevation 1085 m to 1090 m where an effective angle of shearing resistance of 14° has been used. These shear strengths have been used at all elevations except where the design seams apply. Bedding plane shear strengths are listed in Table III. The effective cohesion along the bedding planes is zero.

TABLE II
DESIGN SEAMS FOR SPILLWAY

SEAM NO:	REFERENCE ELEVATION AT THE SPILLWAY CENTRE LINE (m)		DESIGN EFFECTIVE ANGLE OF SHEARING RESISTANCE Ø'
V10	1029	(1030) (1)	14.5°
V12	1035	(1036) (1)	12.5°
V16	1050	(1051) (1)	17.0°
V18	1052.4		16.0°
R1	1068.2		11.5°
R6	1084.5		13.5°
R7	1086.5		13.0°
R8	1087.5		12.5°
R9	1088.5		11.5°
R13	1096.5		15.5°
R14	1099.8		16.0°
R16	1105		13.0°
R17	1107.7		16.5°
R18	1115		12.5°

Note (1) Because of the site-wide and local dip of the strata the elevation of these seams at the flip bucket (given in parenthesis) is slightly different.

TABLE III

BEDDING PLANE SHEAR STRENGTH BETWEEN MAJOR DESIGN SEAMS

ELEVATIONS (m)	REPRESENTATIVE RESIDUAL ANGLE OF SHEARING RESISTANCE ØR	GEOLOGICAL COMPONENT	DESIGN EFFECTIVE ANGLE OF SHEARING RESISTANCE Ø'
Below El.1040	16°	1.5°	17.5°
El.1040 to El.1050	16°	1.5°	17.5°
El.1050 to El.1085	16°	1.5°	17.5°
El.1085 to El.1090	12.5°	1.5°	14.0°
El.1090 to El.1100	16°	1.5°	17.5°
Above El.1100	16°	1.5°	17.5°

3.5.4 Cross Bedding Shear Strength

3.5.4.1 Cross Bedding Shear Strength of Sandstone

The sandstone units contain several combinations of horizontal and vertical joints as discussed in the Preliminary Engineering Report (UMA/Acres (1986)). The cross bedding shear strength assigned to sandstone units is:

Effective angle of shearing resistance = 60°
Effective cohesion = 0

The effective cohesion was assumed to be zero to be consistent with vertical joints that are found in all sandstone units.

3.5.4.2 Cross Bedding Shear Strength of Mudrock

An effective angle of shearing resistance of 35° was assigned to the cross bedding shear strength of all mudrock units. The effective cohesion in cross bedding was varied to reflect the degree of weathering and stress level, and to account for the relative amount of siltstone and claystone in the unit. A higher effective cohesion was used as the unweathered intact claystone content increased.

For the weathered, brecciated, slickensided and highly fractured mudrock layers, the effective cohesion in cross bedding was taken to be zero.

For the unweathered intact mudrock, the effective cohesion in cross bedding shear was assumed to be 200 kPa. For the slightly weathered mudrocks, a cross bedding effective cohesion between 0 and 200 kPa was selected based on the drillhole data.

3.6 MATERIAL PROPERTIES

The material properties listed in Tables II to V were used in the stability analysis and design of the Spillway structures and the excavated slopes.

The selection of these material properties has been based on laboratory testing of soil and rock samples and a geotechnical and geological assessment of the foundation bedrock and the seams found in the rock. These are described in the Oldman River Dam Design Report (UMA/Acres (1988)).

TABLE IV
MATERIAL PROPERTIES

	SANDSTONE	MUDROCK (CLAYSTONE/ SILTSTONE)	LACUSTRINE CLAY	GLACIAL TILL	COMPACTED GRANULAR FILL
Uniaxial Compressive Strength (MPa)	20 - 110	0.8 - 50	---	---	---
Modulus of Elasticity E - (GPa)	2 - 20	0.4 - 1.5	---	---	---
Poissons Ratio	0.3 - 0.4	0.4 - 0.5	---	---	---
Swelling Pressure (kPa)	---	125	---	---	---
Horizontal/Vertical Effective Stress Ratio (K)	0.6	0.6	0.6	0.6	0.4 - 0.5*
Unit Weight (kN/m^3)	23.6	23.6	21.6	21.6	21.6
Coefficient of Permeability, K (cm/sec)	10^{-3}	10^{-5}	10^{-6}	10^{-6}	---

* According to degree of compaction specified.

TABLE V
SHEAR STRENGTH OF SOIL AND CROSS-BEDDING SHEAR STRENGTH IN BEDROCK

	Design Effective Cohesion c (kPa)	Design Effective Angle of Shearing Resistance Ø'	Unit Weight kN/m^3
Compacted Fill			
Zone 1 (impervious)	0	25°	21.6
Zones 2 & 3:(filter & drain) (1)	0	35°	21.6
Zones 4 & 4C: (rock fill)	0	35°	21.6
Zones 5, 5A, 6A, 7 & 8: (granular fill and riprap)	0	35°	21.6
In Situ Soil			
Glaciolacustrine silt and clay	5	22°	21.6
Glacial Till	15	25°	21.6
Bedrock - Headworks, Chute, Flip Bucket, Excavated and Existing Slopes			
Sandstone (cross bedding)	0	60°	23.6
Intact mudrock (cross bedding)	200	35°	23.6
Slightly weathered mudrock (cross bedding)	0 to 200	35°	23.6
Weathered, brecciated, slickensided and badly fractured mudrock (cross bedding)	0	35°	23.6

(1) Gradations for Zone 2 & 3 material are summarized in Table VI.

TABLE VI
GRADATIONS FOR ZONE 2 AND ZONE 3 FILL

Zone 2 Filter

Sieve Size	Percentage by Mass Finer than the Sieve Size
14 mm	100
5 mm	87 - 100
2.5 mm	75 - 90
1.25 mm	55 - 75
0.630 mm	30 - 55
0.315 mm	10 - 35
0.160 mm	2 - 10
0.080 mm	0 - 2

Zone 3 Drain

Sieve Size	Percentage by Mass Finer than the Sieve Size
56 mm	100
40 mm	90 - 100
20 mm	40 - 85
10 mm	15 - 45
5 mm	0 - 20
2.5 mm	0 - 3
0.080 mm	0 - 2

3.7 LATERAL EARTH PRESSURE COEFFICIENTS

The following lateral earth pressure coefficients have been used in the design of structures:

a) Impervious backfill $K = 0.6$, Abutment Monoliths
b) Granular backfill $K = 0.4$ to 0.5 (depending on degree of compaction specified), Chute Wall Monoliths and Flip Bucket.

Fully mobilized active and passive pressures have not been considered in design of the structures.

The lateral earth pressure coefficients have been applied to the effective stresses, and pore pressures have been added to the resultant lateral loads as follows:

$$\sigma_h = K(\sigma v - u) + u$$

Where: σ_h = Horizontal total stress
K = Lateral earth pressure coefficient
σ_v = Vertical total stress
u = Pore water pressure

To account for cohesion tending to decrease the amount of lateral pressure acting on a structure retaining that material, an equivalent coefficient of lateral pressure was determined by modifying the lateral pressure corresponding to a non-cohesive material for cohesion effects.

Plate 4 shows the equivalent coefficient of lateral pressure versus cohesion for two lateral coefficients of 0.6 and 0.5. As shown on this plate, the equivalent coefficient of lateral pressure for a cohesion of 50 kPa and the actual lateral pressure of 0.6 is equal to 0.4.

For retaining walls built close to a stable rock face, with granular fill placed between them, a reduction of the vertical stress in the fill and the horizontal stress acting on the wall was allowed to account for the fill being partly supported by friction on the wall and the rock face.

The Janssen's arching theory (Janssen, 1895) modified by Spangler and Handy (1984) and Frydman and Keissar (1987) was used to calculate the modified coefficient of lateral pressure corrected for the arching effect on the lateral pressure transferred to a rigid structure from a granular fill confined between the structure and the adjacent stable rock face. According to this arching theory the modified coefficient of lateral pressure is as follows:

$$K' = \frac{1}{2 \tan \delta} \; \frac{b}{z} \; [1 - \exp(-2K \frac{z}{b} \tan \delta)]$$

where

K' = modified coefficient of lateral pressure

b = average distance between the wall and the stable rock face

z = depth from wall top

δ = the angle of friction between the wall, the rock and the fill

K = coefficient of lateral pressure

Plate 5 shows the effect of arching on the lateral pressure coefficient for = 20°. As shown on this plate, the arching effect decreases the coefficient of lateral pressure from 0.6 to 0.4.

3.8 ALLOWABLE BEARING PRESSURES

The Spillway structures will be founded on unweathered, undisturbed mudrocks, sandstones and compacted gravels.

The bearing capacity of rock and soil depends on several factors including the applied load, the applied load inclination and eccentricity, the foundation size and depth, the material properties of the bearing strata and the groundwater location. Each individual case was examined and an appropriate bearing capacity was assigned for design.

As a general guide the following allowable net bearing pressures were used:

- Mudrocks	500 kPa
- Channel Sandstone	800 kPa
- Compacted Gravel	200 kPa

A friction angle of 25° was used between the concrete structures and the gravel base material.

3.9 HEAVE AND SETTLEMENT OF STRUCTURES

The construction of the Spillway will involve the excavation of a maximum of approximately 35 m of overburden and rock, the construction of the concrete structures and the placement of backfill behind the structures. This will be followed by impoundment of the reservoir which will cause changes to the groundwater regime. The major regime change wil be saturation of the soil and rock above the existing watertable which will occur approximately one year after completion of the Spillway.

Each of these processes will cause either consolidation settlement or heave of the foundation materials. The effects can be summarized as follows:

a) elastic rebound of bedrock caused by excavation
b) time dependent heave of bedrock due to excavation
c) elastic settlement due to structure loading
d) time dependent settlement due to structure loading
e) time dependent heave due to reservoir impoundment, and
f) time dependent heave caused by a change of state in mudrocks from partly saturated to fully saturated materials

The following assumptions and factors were considered in the analysis of foundation and structure movements:

i) Unloading and reloading consolidation parameters are equal and are independent of the stress state,

ii) Compression Index, Virgin Curve, Cc = 0.072

Compression Index, (Unloading and Reloading) Cr = Cu = 0.031

Initial Void Ratio, e_o = 0.3

Coefficient of Consolidation, C_v = $4.2 \times 10^{-4} cm^2/s$

iii) The bedrock will be initially partially saturated. All the consolidation properties have been however derived from laboratory test results carried out on the saturated samples,

iv) Stress changes resulting from excavation and structure loading were calculated for an assumed finite compressible layer overlying the relatively incompressible and rigid Basal Sandstone.

v) At every vertical section analyzed the actual foundation conditions taken from drill hole data were modelled as mudrock layers sandwiched between sandstone layers. It is assumed that the sandstone layers will act as drainage boundaries. Consolidation and heave will only take place in finite mudrock layers.

vi) Confined swelling pressure and unconfined free swelling strain data were used to estimate swelling caused by change of state in the mudrock.

Based on these assumptions, a total heave of 500mm is estimated at the headworks structure, with an ultimate differential heave of

140mm across the width of the excavation. An ultimate differential heave of 120mm is estimated between the headworks structure and the chute slab.

The time related component of this analysis is uncertain, however, Plate 6 illustrates a prediction of the estimated time-settlement - heave relationship. The progress of heave/settlement will be monitored during construction using various instruments installed in the foundation rock.

3.10 ROCK ANCHORS

Rock anchors will be used in the headworks and flip bucket excavations to stabilize the temporary steep excavations.

The anchors will be Dywidag simple corrosion protected bar anchors and will be installed in 150mm diameter drillholes.

The load capacity of the anchor will be the lesser of the values derived from the rock to anchor grout bond strength, steel to concrete bond strength, steel yield strength and the wedge failure of the rock mass.

The rock to anchor grout working bond strength will be 100 kPa in mudrock and 300 kPa in sandstone.

The anchors will be installed at 30° to the horizontal with a bond length of 6 m located below the critical horizontal sliding plane passing through the toe of the excavation.

4.0 STABILITY

4.1 INTRODUCTION

This section describes the stability analyses carried out for the excavated slopes, headworks, chute and flip bucket.

4.1.1 Stability Analyses

Stability analyses were carried out using the slope stability computer program "PCSTABL4". The program calculates the factor of safety against slope failure by a two dimensional limit equilibrium method. The calculation of the factor of safety is performed by the "Modified Janbu" method of slices which allows the analysis of non-circular composite sliding surfaces as well as those of circular shape. This method is particularly suited to analysis of composite sliding planes which are constrained to follow particular paths such as along seams in bedrock.

4.1.2 Three-Dimensional Factor of Safety

The assumption of two-dimensional, plane strain, geometry is common to all limit equilibrium slope stability calculations, although no landslide ever occurs in plane strain. End effects and the three-dimensional shape of a sliding surface are usually neglected for convenience to simplify the slope stability problem into the limited framework of a two-dimensional model. The importance of three-dimensional effects depends on the geometry of potential sliding surfaces, the type of soil or rock and the presence of adjacent slope constraints. Due to the complex geometry and geology of the excavated and existing slopes at the project site, it is necessary to account for such effects to yield an economical and realistic design.

There is no general method available for three-dimensional analysis of slopes involving sliding surfaces of arbitary shape.

Studies by Baligh and Azzouz (1975) show that for a cohesive soil, consideration of end effects can lead to a 4 percent to 40 percent increase in the factor of safety depending upon the shape of the sliding surface which in turn is a function of the amount of cohesion in the slope material.

Results published by Hovland (1977), Chen and Chameau (1982) and Hungr (1987) indicate that the three-dimensional factor of safety can be from 0 to 40 percent higher than the two-dimensional factor of safety depending on the material properties, the shape of the three-dimensional sliding surface, and the pore pressures.

Hungr (1987) showed that for a material with c = 14.4 kPa and $\phi' = 25°$, the three-dimensional factor of safety is a minimum of 10 percent greater than the two-dimensional factor of safety when the width of sliding mass is equal to or less than 2.5 times the height of slope. The ratio of the three-dimensional factor of safety to the two-dimensional factor of safety increases as the cohesion increases. From the results given by Hungr (1987) this ratio becomes equal to 1.15 when the cohesion is 28.7 kPa and the width to height ratio of the sliding mass is equal to 2.5.

The following procedure was used to investigate the effect of three-dimensional geometry on the factor of safety of the spillway headworks and chute and their foundations. The two-dimensional stability of a number of parallel, vertical planes, equally spaced along the length of an assumed potential sliding mass was calculated. The resulting factors of safety were then averaged and compared with the factor of safety of the vertical plane through the deepest part of the mass. The average factor of safety was considered to approximate the three-dimensional factor of safety. The ratio of the average (three-dimensional) to deepest (two-dimensional) factor of safety was found to be 1.4 for the headworks structure stability and to be 1.15 for the chute excavation slopes.

Based on these results and those given by Hungr (1987) it was concluded that it would be conservative to assume a ratio of three-dimensional to two-dimensional factors of safety of 1.1 for the design of the structures and excavated slopes, where the three-dimensional effect is applicable.

4.2 STABILITY OF EXCAVATED SLOPES

4.2.1 General

The excavated slopes include those along the spillway approach channel, chute and flip bucket. The locations of the slopes analysed are shown on Plate 7.

The stability analyses have considered deep seated failure surfaces only. Slaking of the mudrocks in the slopes will cause local surficial sloughing of all bedrock slopes during the life of the project. The design of excavated slopes and analysis of existing slopes adjacent to structures that will not be excavated has taken into account the long term effects of slaking. Protection has been provided where necessary to prevent detrimental erosion.

4.2.2 Geology

The excavated slopes are in the Upper Mudrock sequence overlain by Quarternary surficial deposits of glacial till and glaciolacustrine silt and clay.

A simplified stratigraphy for each slope cross section analyzed was developed from nearby drillhole and surface exposure data. The stratigraphy is generally composed of glaciolacustrine silt and clay, clay till, weathered mudrock, slightly weathered mudrock, intact mudrock and sandstone. Mudrock includes both siltstone and claystone. In locations where the mudrock is composed of several thin zones of interbedded weathered and intact rocks, it was categorized as being slightly weathered. An appropriate effective cohesion intercept was assigned to the shear strength of these zones to reflect the relative amount of weathered and unweathered rock and the proportions of sandstone, siltstone and claystone in the zones.

Several subhorizontal major seams exist in the mudrock as discussed in Section 3. These seams were considered in the analyses by modelling a flat segment of the sliding planes to pass through them.

A small normal fault plane was found in the flip bucket area. It trends approximately north-south, dips 50° west. A discontinuous clay gouge up to 6 mm thick was found on the fault plane. Approximately 350 mm of dip-slip offset was found. Physically it resembles an inclined joint plane of through-going continuity, which could lead to local instability of unfavorable orientated excavation cuts.

If, during construction other similar features are encountered, their site specific aspects will be obtained from geological mapping.

Appropriate design modifications will then be implemented which may include:

- Local modification of excavation limits to avoid the feature.
- Removal of unstable materials.
- Additional rock support and stabilization measures.

4.2.3 Material Properties for Stability Analyses

The shear strength and unit weight parameters used in the analyses are listed in Tables II to V inclusive. These parameters have been derived from geotechnical and geological assessments of the bedrock and seam properties and the results of the laboratory testing program as described in the Oldman River Dam Design Report (UMA/Acres (1988)).

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Overburden

The effective angle of shearing resistance and the unit weight assigned for the glaciolacustrine silt and clay and clay till have been based on the index and shear strength test results as discussed in the Preliminary Engineering Report, UMA/Acres (1986). A small effective cohesion intercept was used in the slope stability analyses to establish circular sliding planes entirely in the overburden. The shear strength of the overburden is given in Table V.

Bedrock

Three sets of shear strength parameters are required for the bedrock stability analysis. These comprise the shear strength of the rock across the bedding planes, the shear strength in the seams along the bedding planes and the shear strength along the bedding planes in the intact rock between the seams.

The cross-bedding effective angle of shearing resistance selected for each of the bedrock types was based on the laboratory test results as discussed in the Preliminary Engineering Report, UMA/Acres (1986), the Oldman River Dam Design Report (UMA/Acres (1988)) and Section 3.5.4. An effective cohesion of the cross-bedding shear strength of some of the mudrocks was used in stability analyses. The effective cohesion was selected on the basis of degree of weathering and rock type. For the sandstone, and for weathered, brecciated, slickensided and highly fractured mudrock zero effective cohesion was assumed. For the unweathered intact mudrock an effective cohesion of 200 kPa was used. For the slightly weathered mudrocks, the assigned effective cohesion varied between 0 and 200 kPa depending on the degree of weathering and on the relative amount of siltstone and claystone recorded in the adjacent drillholes and surface exposures. The cross-bedding shear strengths are given in Table V.

The shear strength in the seams along the bedding planes and the shear strength along the bedding planes between the seams were discussed in Section 3. The effective cohesion along all seams is zero. Their values are given in Tables II and III.

4.2.4 Pore Pressure Parameters

The phreatic surface for the steady state seepage condition was determined from the finite element analyses of seepage through the abutments described in the Oldman River Dam Design Report (UMA/Acres (1988)).

The permeability of the overburden and bedrock is likely to be low in extensive local zones and water will drain slowly from the slopes in these areas. Drainage will be primarily through fissures in the clay till and through weathered and fractured zones in the bedrock. The phreatic surface for the rapid drawdown condition was therefore assumed to be close to the steady state condition at full supply level within the slope and assumed to generally follow the slope some distance in from the surface to allow for some pore pressure dissipation. The actual phreatic surfaces assumed are shown on Plates 8 to 14 inclusive.

4.2.5 Method of Analysis

The slope failure mechanisms considered most appropriate for the topography and rock and soil types found at the project site are:

- Sliding on a near circular surface where the sliding surface passes entirely in overburden.
- Sliding on a composite surface through the overburden and bedrock, with the horizontal segment of the sliding surface passing through the bedrock and overburden contact.
- Sliding on a composite surface through the overburden and bedrock or through the bedrock only, where the overburden is not present, with the horizontal segment of the sliding surface passing through a seam parallel to the bedding planes.

Sensitivity analyses were done for sections using sliding surface passing through seams above, at and below the toe of the slope.

These analyses showed that the critical sliding surface always passes through the toe of the slope. The stability was also checked for higher seams where lower effective angles of shearing resistance existed.

The slopes have been designed for the required minimum safety factors given in Table VII.

TABLE VII
REQUIRED MINIMUM FACTORS OF SAFETY

STABILITY CASE	REQUIRED MINIMUM FACTOR OF SAFETY
1. Reservoir at full supply level (FSL) and steady state seepage conditions.	1.5
2. Reservoir at FSL and steady state seepage with horizontal pseudo-static seismic loading 0.065 g.	> 1.0
3. Rapid reservoir drawdown from (FSL) to minimum supply level.	1.3
4. Rapid reservoir drawdown from FSL to minimum supply level with horizontal pseudo-static seismic loading 0.065 g (for upstream slopes only).	> 1.0

4.2.6 Results of Analysis

4.2.6.1 Approach Channel

Plate 8 illustrates the sections analyzed and the factors of safety obtained. Section A is the deepest section in the approach channel with sliding surfaces through seams R14, R15, R16 and R17 at elevations 1099.8, 1102, 1105 and 1107.7 m. The critical sliding surface passes through seam R14 at elevation 1099.8 at the toe of the excavation.

To obtain the required minimum factors of safety the approach channel rock slopes will be excavated to a slope of one horizontal to one vertical with a 10 m berm at elevation 1120 m, with a 10 m berm at the rock-overburden contact and a slope of three horizontal to one vertical in the overburden.

Plate 9 shows the relationship between the factor of safety and the effective angle of shearing resistance of a seam.

4.2.6.2 Spillway Chute

The spillway chute side slopes have been analyzed by considering three sections along the chute as shown on Plate 7. The location of the phreatic surface and stratigraphy varied at each section and Plates 10 through 15 illustrate the results obtained. In all cases the steady state seepage case governed. The one degree site-wide dip of the bedrock in the chute area was included in the analyses.

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The analysis dictated an excavation slope of one horizontal to one vertical in the rock, with a 10 metre-wide bench at every 20 metres height from the base of the excavation, with a 10 metre-wide bench at the rock-overburden contact, and a slope of three horizontal to one vertical in the overburden above.

The three sections analysed were as follows:

Section B

- approximately 50 m downstream of the headworks
- critical sliding surface passes through Seam R16 with $\phi' = 13°$ at elevation 1105 m as shown on Plate 10
- Plate 11 indicates the sensitivity of factor of safety vs. effective angle of shearing resistance.

Section C

- approximately 140 m downstream of the headworks
- critical sliding surface passes through seam R9 with $\phi' = 11.5°$ at elevation 1088.5 m as shown on Plate 12
- Plate 13 indicates the sensitivity of factor of safety vs. effective angle of shearing resistance.

Section D

- approximately 230 m downstream of the headworks
- critical sliding surface through seam R1 with $\phi' = 11.5°$ at elevation 1068.2 m as shown on Plate 14
- Plate 15 indicates the sensitivity of factor of safety vs.

effective angle of shearing resistance.

For the slopes in the region of Section B the calculated safety factor based on a two dimensional analyses is about seven percent less than the required minimum. However, when the following factors are taken into account the actual safety factor will be greater than the required minimum.

a) The spillway chute is inclined at a slope of approximately 6.4 horizontal to one vertical along the base of the excavated slope. The seams are essentially horizontal and any slide through them will terminate at the upstream end where the seam lies below the bottom of the excavation. As the ground surface also slopes downwards in the same direction as the spillway chute excavation, the seam effectively daylights downstream. These constraints will limit the lateral extent of any slide and will cause the actual safety factor to be greater than that calculated.

b) The cohesion present in the mudrock will also increase the factor of safety through lateral restraint.

c) The phreatic surface assumed in the analyses is conservative in considering the combined effect of the grout curtain, pressure relief drains, the drainage blanket beneath the chute slab and river slope on the back side of the excavated slope.

d) The three-dimensional effect of the failure block can be assumed to increase the safety factor by at least 10 percent as discussed in Section 4.1.2.

4.2.6.3 Flip Bucket

The flip bucket excavation side slopes are similar to those at the spillway chute. The slopes will be excavated to one horizontal to one vertical in the rock, a 10 metre-wide bench every 20 metres height from the base of the excavation, a 10 metre-wide bench at the rock-overburden contact, and a slope of three horizontal to one vertical in the overburden above.

During the construction period, the lower section of the flip bucket will be excavated to a steep slope that will be temporarily stabilized using a shotcrete tie-back wall and deep seated rock anchors. The stability analysis of the slopes and the anchored tie-back wall are discussed in Section 4.6.

4.3 HEADWORKS STABILITY

4.3.1. Introduction

This section describes the stability analyses carried out for the foundation and the structural elements of the headworks for the established load combinations to ensure compliance with the specified stability criteria. The sensitivity of the factor of safety to variations in the angle of effective shearing resistance of the rock was also assessed. The general arrangement of the headworks is shown on Plate 16.

4.3.2 Seepage Analysis

Seepage flow analyses were carried out to establish the variation in porewater pressure within the rock foundation in conjunction with the foundation stability analyses.

A two dimensional finite element analysis of seepage flows through porous media was carried out to determine the steady state seepage and the associated variation of the pore pressure in the rock foundation beneath the headworks. The reservoir level was assumed to be at FSL with the gates closed.

The following conditions were considered in assessing the effects on pore pressure variation:

- variable grout curtain depths
- effective and ineffective grout curtain
- effective and ineffective pressure relief drain system (drainage curtain) including the drainage tunnel
- crack development between the upstream face of the ogee monolith and the rock (thus allowing full reservoir head to act on the base of the monolith).

Seven combinations of grout curtain, drainage system effectiveness, and crack development were considered as summarized in Table VIII. The results for Case 1 are shown on Plate 17 in which the pore pressure variation is presented in the form of equipotential lines showing the percentage change from full reservoir head.

TABLE VIII
COMBINATION OF DRAINAGE FEATURES INCLUDED IN THE SEEPAGE ANALYSIS

CASE	EFFECTIVE GROUT CURTAIN	EFFECTIVE DRAINAGE TUNNEL	CRACK DEVELOPMENT
1	NO	NO	NO
2	NO	YES	NO
3	YES	YES	NO
4	NO	NO	YES
5	NO	YES	YES
6	YES	YES	YES
7	YES*	YES	YES

* Relatively permeable grout curtain

The results show that the configuration of the sloping spillway chute is ineffective in reducing the uplift pressure acting on the base of the headworks structure for even the worst scenario of ineffective grout curtain, blocked drainage system and upstream crack development.

4.3.3. Foundation Stability

The foundation of the spillway headworks structure consists of interbedded sandstones, siltstones and claystones. Several subhorizontal seams exist at various elevations and these control the stability.

Foundation stability was analyzed considering the two dimensional limit equilibrium of the structure and underlying bedrock, for potential failure planes passing through seams R9, R13 and R14, at elevations 1088.5m, 1096.5m and 1099.8m respectively.

A cross bedding effective angle of shearing resistance of 35° and a cross bedding effective cohesion of 50 kPa were assigned to the mudrock between the Spillway headworks structure and seams R9, R13 and R14.

Pore water pressures and seepage will be reduced by means of the grout curtain and drainage system installed in the rock beneath the structure. In the analyses, four different pore water pressure distributions were considered along the failure plane. These are described below and are shown on Plate 18.

Case 1 Pore water pressure equivalent to:

- at the upstream heel of the structure
  - full supply level with linear hydrostatic variation with depth to the elevation of the assumed seam;
- at downstream toe of the structure
  - tailwater level with linear hydrostatic variation with depth;

- at the drain location
  - tailwater level plus one third of the difference between the full supply level and the tailwater level
- between the upstream heel to the drain location
  - linear variation along the seam from the full supply level to the tailwater level plus one third of the difference between the full supply level and the tailwater level
- between the drain location and the downstream toe of the structure
  - linear variation from the tailwater level plus one third of the difference between the full supply level and the tailwater table to the tailwater level.

Case 2 Pore water pressures from the finite element analysis of steady state seepage, assuming that the grout curtain is effective and drains are operational. Between the drains and the toe, the phreatic surface was assumed to be at the tailwater level.

Case 3 Pore water pressure equivalent to:

- the full supply level at the upstream heel of the structure with linear hydrostatic variation with depth;
- the tailwater level at the downstream toe of the structure with linear hydrostatic variation with depth;
- linear variation from the full supply level to the tailwater level between the heel and the toe of the structure.

This presents a case where a continuous crack exists below the structure to the depth of the failure plane, with the grout curtain ineffective and the drains inoperative.

Case 4 Pore water pressure from the finite element analysis of steady state seepage discussed in Section 4.3.2, assuming an ineffective grout curtain and inoperative drains.

Results of the analyses carried out for different values of shear strength of the seams R9, R13 and R14, are shown on Plates 18, 19 and 20 and Tables IX and X and XI respectively. Adequate factors of safety are provided for all cases except for the failure plane passing through seam R9 at elevation 1088.5 m. However, in all these analyses, only two dimensional conditions were considered. Instability of the concrete headworks structure and its foundation rock would involve failure on a plane passing through one of the seams beneath the structure and extending laterally through the overlying rock strata up to ground surface. As such a failure plane would mobilize significant cross bedding shear strength it is reasonable to consider the three dimensional conditions and to increase the safety factor by a minimum of 10 percent as discussed in Section 4.1.2. The factor of safety against sliding along the critical seam R9 will then be acceptable as indicated in Table IX.

TABLE IX
HEADWORKS FOUNDATION STABILITY ANALYSIS
LIMIT EQUILIBRIUM METHOD
SLIDING ALONG SEAM R9 AT EL. 1088.5 m

ALONG BEDDING SHEAR STRENGTH OF SEAM
$\emptyset' = 11.5^{\circ}$, $c' = 0$

CROSS BEDDING SHEAR STRENGTH IN MUDROCK BETWEEN
STRUCTURE SURFACE AND SEAM
$\emptyset' = 35^{\circ}$, $c' = 50$ kPa

PORE WATER * PRESSURES ALONG FAILURE PLANE	RESERVOIR AT FSL GATES CLOSED		RESERVOIR AT FSL GATES CLOSED HORIZONTAL SEISMIC LOADING OF 0.065 g	
	REQUIRED FACTOR OF SAFETY	** FACTOR OF SAFETY AGAINST SLIDING	REQUIRED FACTOR OF SAFETY	** FACTOR OF SAFETY AGAINST SLIDING
CASE 1	1.5	1.5	>1.0	1.02
CASE 2	1.5	1.5	>1.0	1.05
CASE 3	1.3	1.3	-	-
CASE 4	1.3	1.4	-	-

* PORE PRESSURES ALONG FAILURE PLANE (SEE PLATE 18)

CASE 1: HEAD DROPS LINEARLY FROM FSL AT HEEL TO (TWL + 1/3 [FSL - TWL]) AT DRAINS AND TO TWL AT TOE

CASE 2: FINITE ELEMENT SEEPAGE ANALYSIS, EFFECTIVE GROUT CURTAIN, DRAINS OPERATIONAL

CASE 3: LINEAR VARIATION FROM FSL AT HEEL TO TWL AT TOE

CASE 4: FINITE ELEMENT SEEPAGE ANALYSIS, INEFFECTIVE GROUT CURTAIN, DRAINS INOPERATIVE

** THESE FACTORS OF SAFETY ARE BASED ON THE RESULTS OF 2-DIMENSIONAL STABILITY ANALYSIS PLUS A 10% ADJUSTMENT TO INCLUDE THE 3-DIMENSIONAL EFFECT.

TABLE X
HEADWORKS FOUNDATION STABILITY ANALYSIS
LIMIT EQUILIBRIUM METHOD
SLIDING ALONG SEAM R13 AT EL. 1096.5 m

ALONG BEDDING SHEAR STRENGTH OF SEAM
$\phi' = 15.5^{\circ}$, $c' = 0$

CROSS BEDDING SHEAR STRENGTH IN MUDROCK BETWEEN
STRUCTURE SURFACE AND SEAM
$\phi' = 35^{\circ}$, $c' = 50$ kPa

PORE WATER * PRESSURES ALONG FAILURE PLANE	RESERVOIR AT FSL GATES CLOSED		RESERVOIR AT FSL GATES CLOSED HORIZONTAL SEISMIC LOADING OF 0.065 g	
	REQUIRED FACTOR OF SAFETY	FACTOR OF SAFETY AGAINST SLIDING	REQUIRED FACTOR OF SAFETY	FACTOR OF SAFETY AGAINST SLIDING
CASE 1	1.5	1.8	>1.0	1.3
CASE 2	1.5	1.9	>1.0	1.3
CASE 3	1.3	1.6	-	-
CASE 4	1.3	1.7	-	-

* PORE PRESSURES ALONG FAILURE PLANE (SEE PLATE 18)

CASE 1: HEAD DROPS LINEARLY FROM FSL AT HEEL TO (TWL + 1/3 [FSL - TWL]) AT DRAINS AND TO TWL AT TOE
CASE 2: FINITE ELEMENT SEEPAGE ANALYSIS, EFFECTIVE GROUT CURTAIN, DRAINS OPERATIONAL
CASE 3: LINEAR VARIATION FROM FSL AT HEEL TO TWL AT TOE
CASE 4: FINITE ELEMENT SEEPAGE ANALYSIS, INEFFECTIVE GROUT CURTAIN, DRAINS INOPERATIVE

TABLE XI
HEADWORKS FOUNDATION STABILITY ANALYSIS
LIMIT EQUILIBRIUM METHOD
SLIDING ALONG SEAM R14 AT EL. 1099.8 m

ALONG BEDDING SHEAR STRENGTH OF SEAM
$\phi' = 16^{\circ}$, $c' = 0$

CROSS BEDDING SHEAR STRENGTH IN MUDROCK BETWEEN
STRUCTURE SURFACE AND SEAM
$\phi' = 35^{\circ}$, $c' = 50$ kPa

PORE WATER * PRESSURES ALONG FAILURE PLANE	RESERVOIR AT FSL GATES CLOSED		RESERVOIR AT FSL GATES CLOSED HORIZONTAL SEISMIC LOADING OF 0.065 g	
	REQUIRED FACTOR OF SAFETY	FACTOR OF SAFETY AGAINST SLIDING	REQUIRED FACTOR OF SAFETY	FACTOR OF SAFETY AGAINST SLIDING
CASE 1	1.5	1.9	>1.0	1.4
CASE 2	1.5	2.1	>1.0	1.4
CASE 3	1.3	1.7	-	-
CASE 4	1.3	1.8	-	-

* PORE PRESSURES ALONG FAILURE PLANE (SEE PLATE 18)

CASE 1: HEAD DROPS LINEARLY FROM FSL AT HEEL TO (TWL + 1/3 [FSL - TWL]) AT DRAINS AND TO TWL AT TOE
CASE 2: FINITE ELEMENT SEEPAGE ANALYSIS, EFFECTIVE GROUT CURTAIN, DRAINS OPERATIONAL
CASE 3: LINEAR VARIATION FROM FSL AT HEEL TO TWL AT TOE
CASE 4: FINITE ELEMENT SEEPAGE ANALYSIS, INEFFECTIVE GROUT CURTAIN, DRAINS INOPERATIVE

4.3.4 Stability of Ogee and Abutment Monoliths

The stability of individual ogee and abutment monoliths in the longitudinal direction of the spillway structure has been established on the basis of the forces equilibrium method by considering sliding along a bedding plane just below the top of the excavated rock with an effective angle of shearing resistance of 17.5°.

A pressure relief drainage system and a grout curtain are incorporated to control seepage and to reduce uplift pressure.

Each ogee monolith was analyzed as an independent structure without the beneficial effects of the converging geometry of the headworks structure and the presence of the shear keys at logitudinal joints. In the transverse direction (parallel to the ogee centreline), lateral stability of the abutment monolith was based on interaction between individual monoliths. The lateral loading was considered to be resisted by the headworks structure acting as a monolithic unit with the ogee monoliths acting as a strut between the abutment monoliths.

The stability of individual monoliths has been analyzed using combinations of the following loads:

i) Dead Loads

The gravity loading from the ogee and abutment monoliths, piers, bridge structure, control gate, tower structure with hoist equipment, rock mass contained within the rigid body defined by the failure planes and the mass of water supported by the ogee base.

ii) Lateral Water Loads

The hydrostatic pressure loads due to the reservoir water level acting on the gate, piers and upstream face of the ogee and abutment monoliths.

iii) Lateral Ice Load

The lateral thrust due to the thermal expansion of an ice sheet represented by a uniformly distributed line load of 150 kN/m acting on the upstream face of the ogee and abutment monoliths and on the gates at elevation 1118.3m.

iv) Lateral Rock and Earth Loads

The lateral loading due to the lateral rock pressure acting on the upstream face of the ogee and abutment monoliths and the lateral pressure from the chute slab gravel acting on the downstream face of the ogee and abutment monoliths. A lateral pressure coefficient of 0.6 was used for both rock and gravel.

v) Uplift Loads

The effect of uplift pressure distribution along the base of a monolith corresponding to both operating and fully blocked conditions of the drains were assessed. In the case of operating drains, a 67% drain efficiency was assumed. The corresponding uplift pressure distribution is a bi-linear pressure variation along the base defined by the reservoir level pressure at the upstream face, a pressure corresponding to the tailwater pressure plus 33% of the net pressure head at the location of the drainage curtain and the tailwater elevation of 1101.8m was assumed.

The blocked drain uplift pressure distribution was taken as a linear pressure variation along the base defined by the reservoir level pressure at the upstream face and the tailwater pressure at the downstream face.

vi) Earthquake Load

An earthquake load equivalent to a minimum horizontal ground acceleration of 0.065g was used.

The equivalent lateral inertia force was applied to the mass of the ogee and abutment monoliths, piers, bridge structure and enclosed rock mass as well as the gate and the gate hoist structure. The lateral force from the hydrodynamic pressure due to the reservoir at FSL was taken into account. Results of the stability analyses for Loading OC-3 are shown on Table XII.

The factors of safety against sliding failure were determined on the basis of treating the monolith and the associated rock within the defined failure planes as a rigid body. The corresponding overturning stability compatible with the assumption of rigid body behaviour is thus based on the assumption of linear vertical pressure variation acting on the base of the structure. In view of the assumed rigid body behaviour and the relatively low level of ground acceleration, the pseudo-static method of analysis was used to determine the effect of the earthquake loading.

For the assessment of sliding and overturning stability on a failure plane at the base of the monolith the three dimensional shape of the monolith and the associated loading were used in the analysis to take into account the trapezoidal shape of the individual monolith, as shown on Plates 21 and 22. The conventional two-dimensional strip method of analysis would yield less conservative results.

The failure surface is defined by an assumed vertical crack in the rock at the upstream face of the monolith, that extends down to a horizontal sliding surface at the structure/bedrock interface at elevation 1100 m.

4.3.4.1 Ogee Monoliths

Stability analyses to establish the factor of safety against sliding, flotation and overturning of the typical ogee monolith were performed for several critical loading cases to verify compliance with the specified stability criteria. The results of the analyses identifying the load cases considered, the governing criteria and the factor of safety against the occurrence of the various modes of stability failure are shown in Table XII.

STABILITY

The distribution of net sustained bearing pressure transmitted to the rock foundation at the base of the typical ogee monolith was calculated for the following two load cases:

- reservoir at FSL with uplift pressure corresponding to operating drains,

- dry condition represented by empty reservoir and no uplift pressure.

The results obtained for the extreme values of net bearing pressures at the upstream and downstream face of the ogee monolith for each case are shown in Table XIII.

TABLE XII

OGEE MONOLITH

STABILITY ANALYSIS - FORCES EQUILIBRIUM METHOD

	LATERAL STABILITY		FLOTATION		OVERTURNING STABILITY		
LOADING CONDITION	CALCULATED FACTOR OF SAFETY	CRITERION	CALCULATED FACTOR OF SAFETY	CRITERION	CALCULATED ECCENTRICITY (m)	CRITERION L/6 OR L/4 (m)	COMMENTS
Operating Conditions							
Reservoir at FSL Drains Operating Ice Loading (OC-1) (1)	1.53	1.5	5.1	1.5	1.44	6.67	Detailed Calculation
Reservoir at FSL Drains Blocked Ice Loading (OC-2)	1.25	1.25	2.9	1.25	2.7	6.67	Detailed Calculation
Reservoir at FSL Drains Operating Earthquake Loading (OC-3)	1.16	>1.0	4.4	1.0	2.24	10.0	Detailed Calculation
Reservoir at FSL Earthquake Loading 1 in 5 yr Flow 600m^3/s (OC-4)	> 1.0	1.0	> 1.0	1.0	<< 10.0	10.0	By Inspection
Reservoir Drawdown from FSL Drains Blocked (OC-5)	1.97	1.5	2.4	1.5	5.82	6.67	Detailed Calculation
Spillway Design Flood Reservoir at FSL Drains Operating (OC-6)	> 1.5	1.5	>1.5	1.5	1.5	<6.67	By Inspection
Severe Conditions							
PMF Loading Drains Inoperative (SC-1)	1.4	>1.0	2.7	1.15	<< 10.0	10.0	Approximate Calculation

Note: (1) These loading conditions are described in more detail in Section 4.5.2.

TABLE XIII
NET BEARING PRESSURE DISTRIBUTION UNDER TYPICAL OGEE MONOLITH FROM SUSTAINED LOADS

Load Condition	NET BEARING PRESSURE (kPa)	
	UPSTREAM FACE	DOWNSTREAM FACE
Reservoir at FSL with Drains Operating	200	274
Dry Structure, Empty Reservoir and no Uplift	208	265

4.3.4.2 Abutment Monoliths

The abutment monoliths include the first split pier, the first ogee section and a gravity type retaining structure which are designed to resist hydraulic, ice, earth and rock pressure, uplift, bridge and superstructure loading.

The overall size and shape of the structure was dictated by the functional requirements of the approach wall curve, the ogee shape and the gate dimensions.

The concrete mass was distributed to achieve an optimal uniform bearing pressure to avoid large differential settlements. The maximum allowable bearing pressure was limited to 500 kPa. See Plate 22.

The stability in the direction of flow matches the standard center ogee section. For the stability in the transverse direction, to resist the backfill pressures, the strutting action of the base slab is required.

A lateral earth pressure coefficient of $k = 0.6$ was assumed in calculation of lateral earth pressures on the structure.

4.3.5 Approach Walls

The approach walls consist of reinforced concrete semi-gravity retaining walls designed for the following design parameters:

Soil Surcharge 0.6 m

Coefficient of Lateral Earth Pressure K = 0.6
Effective Angle of Sliding Resistance = 17.5°
Safety Factor Against Sliding = 1.5

4.4 CHUTE STABILITY

4.4.1 Chute Wall Monoliths

The chute wall stability has been analyzed for the following loading combinations:

i) Construction Conditions

CC-1
- no water in chute
- backfill in place (dry)
- surcharge 0.3 m soil

ii) Operating Conditions

Reservoir at FSL

OC-1
- Gates closed
- No water in chute
- Phreatic line in earth fill at maximum gradient under full reservoir conditions
- Backfill in place

OC-2
- Water surface corresponding to Spillway Design Flood
- Backfill in place

OC-3 - Case OC-1 plus .065g horizontal seismic acceleration

OC-4 - 1 in 5 year discharge of 600 m^3/sec.
- Backfill in place
- .065g horizontal seismic loading

iii) Severe Conditions

PMF

SC-1 - Rapid closure of gates following PMF
- no water in chute
- Phreatic line corresponding to PMF conditions
- Backfill in place

SC-2 - Water surface corresponding to PMF
- Backfill in place

4.4.2 Stability Criteria

The following stability criteria was used:

- For Monoliths 1, 2, 3 Design: $\phi' = 17.5°$,
  K = 0.5,
  Weak seams R13 and R14 will be exposed and removed from beneath the monolith during construction.
- For Monoliths 4 to 23: Design $\phi' = 14.0°$ on rock
  Design $\phi' = 25.0°$ on gravel
  K = 0.5
  Weak seams R1, R6, R7, R8 and R9 will be exposed and removed from beneath the monolith during construction.

4.4.3 Governing Load Cases

Table XIV summarizes the results of the stability analyses for the governing cases.

Stability of Monoliths 1, 2 and 3 was governed by Load Case CC-1. Seams R14 at elevation 1099.8 (Ø' = 16.0°) and R13 at elevation 1096.5 (Ø' = 15.5°) will be removed from beneath the monolith during construction as indicated on Plate 23.

Stability of Monoliths 4 to 23 was governed by Load Case SC-2. Seams R1 at elevation 1068.2 (Ø' = 11.5°), R6 at elevation 1084.5 (Ø' = 13.5°), R7 at elevation 1086.5 (Ø' = 13.0°), R8 at elevation 1087.5 (Ø' = 12.5°) and R9 at elevation 1088.5 (Ø' = 11.5°) will be removed from beneath the monolith during construction as indicated on Plate 24.

TABLE XIV
CHUTE WALL STABILITY

CASE CC-1	MONOLITH 1	2	3	4 to 22	23
Bearing Pressure (kPa)	473	405	363	170(rock) 147(Gravel)	228(rock) 174(Gravel)
Allowable Bearing Pressure (kPa)	500	500	500(rock)	200(Gravel)	200(Gravel)
Factor of Safety - Sliding	1.5	1.5	1.5	1.65	1.93
Allowable F of S - Sliding	1.5	1.5	1.5	1.5	1.5
Location for Overturning	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3
Allowable Location for Overturning	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3

CASE SC-2	MONOLITH 1	2	3	4 to 22	23
Bearing Pressure (kPa)	390	367	298	200(gravel) 267(Rock)	181(Gravel) 248(Rock)
Allowable Bearing Pressure (kPa)	650	650	650(rock)	250(Gravel)	250(Gravel)
Factor of Safety - Sliding	2.13	1.93	1.72	1.22	1.24
Allowable F of S - Sliding	1.2	1.2	1.2	1.2	1.2
Location for Overturning	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3	Middle 1/3
Allowable Location for Overturning	Middle 1/2	Middle 1/2	Middle 1/2	Middle 1/2	Middle 1/2

4.5 FLIP BUCKET STABILITY

The flip bucket consists of a concrete gravity structure placed on a stepped rock foundation, as shown on Plate 25.

4.5.1. Supplementary Foundation Investigation

As described in Section 3.5.2 because of difficulties in obtaining intact core samples from drillholes, the design of the flip bucket was based on seam location and parameters inferred from the drillhole data and from other data on the seams obtained in the Dam foundation some 500 m away. An exploratory shaft investigation was scheduled for May and June, 1989 after the award of the Spillway contract and after the area had been excavated by the Dam contractor to within 2 to 12 m of final foundation grade. The Spillway contract was awarded on the basis of the interim design but a clause was included in the contract to allow certain specified changes to be made to the flip bucket design after the award of contract, should substantially different foundation conditions be discovered.

The objectives of the supplementary investigation were:

- to determine in situ the location, condition and continuity of seams R10 and R12 inferred from drillhole data and any other seams that may be present;
- to determine in situ the type and condition of the rock in which the seams are located, and
- to measure the shear strength of the seams.

STABILITY

The location of the two shafts are shown on Plate 26 together with the excavated profiles included in the Dam and Spillway contracts. Both seams V10 and V12 were found to be within a few centimetres of their inferred locations. These are also shown on Plate 26. The continuity of these seams was also confirmed.

Core samples 200 mm in diameter were taken from both seams and shear box test were done on 100 mm by 100 mm specimens of the seams cut from the cores. The results are plotted together with others from the same seams sampled in the Dam foundation on Plates 27 to 30. Plates 27 and 29 show the residual angles of shearing resistance and Plates 28 and 30 show the peak angles of shearing resistance. The latter values were used together with information on the in situ condition of the seams to assign a geological component to be added to the residual shear strength to produce the values to be used in design. This process is summarized in Section 3.5.2 and is described in detail in the Dam Design Report. Plates 27 and 29 show the residual angles of shearing resistance that were considered to be representative of the test results and that were used in the interim design. It can be seen that all of the test results from the seam samples taken from the flip bucket shafts lie close to but above the representative residual angle of shearing resistance. A similar trend was found with the laboratory peak of in situ values plotted on Plates 28 and 30. It was therefore considered that the interim values should be used for final design.

The effective angle of shearing resistance used in the design for seam V10 at elevation 1030 m is 14.5° which is made up of a representative residual value of 13.5° (from Plate 27) plus 1.0°

for the geological component. The latter takes into account the roughness or non-planarity of the seam in situ and the portion of the peak shear strength of the seam material still available. Similarly the effective angle of shearing reistance used in the design for seam V12 at elevation 1036 m is 12.5°. This is made up of a residual value of 12° from the laboratory tests plus 0.5° to account for the roughness of the seam in situ.

4.5.2 Stability Analyses

The stability of the flip bucket and its foundation was analysed primarily by the limit equilibrium method for sliding on surfaces passing along seams V10 and V12 which were assumed to be at elevations 1030 m and 1035 m respectively. It was subsequently found during the 1989 flip bucket foundation investigation that in the vicinity of the flip bucket seam V12 is actually at elevation 1036 m and will therefore be excavated at the bottom step as shown on Plate 25. As a consequence of this the factors of safety reported in this section are conservatively low. The stability of the structure was also checked using the forces equilibrium method for sliding along a surface that follows the concrete-rock interface down the steps and then along seam V12 below the bottom step.

Limit Equilibrium Method

In the stability analysis of the flip bucket an effective cross bedding angle of shearing resistance of 60° was used for sandstone between elevations 1035 m and 1052 m, and an effective cross bedding angle of shearing resistance of 35° and effective cohesion of 50 kPa were used for mudrock between elevations 1052 m and 1059 m. These and other material properties and parameters are listed on Plate 31.

The following loading conditions were analyzed:

Construction Conditions

CC-1 - End of excavation, no structure
- Backwater elev. 1042 m, Tailwater elevation 1035 m.

CC-2 - End of Excavation, no structure
- CC-1 plus horizontal seismic loading = .065g

CC-3 - End of Construction
- Max. pore water pressure ru = 0.3. Backwater level at elevation 1051.2m, constant for 50 percent of the flip bucket, then dropping to the tailwater level at elevation 1035m.

CC-4 - End of Construction
- CC-3 plus horizontal seismic loading = .065g

CC-5 - End of Construction
- Max. porewater pressure ru = 0.3. Backwater level at elevation 1051.2m, constant for 50 percent of the flip bucket, then dropping to the tailwater level at elevation 1050m.

CC-6 - End of Construction
- CC-5 plus horizontal seismic loading = .065g

Operating Conditions

OC-1 - Normal Condition
- No flow, Backwater and Tailwater at elev. 1050

OC-2 - 1 in 5 year discharge - 600 m^3/sec.
- Backwater and Tailwater at elev. 1052.4
- Horizontal seismic loading = .065g

OC-3 - Design Flood
- Backwater at elev. 1058m, Tailwater at elev. 1057.0

OC-4 - Rapid Drawdown from Design Flood
- Backwater at elev. 1056.0, Tailwater at elev. 1050.0

Severe Conditions

SC-1 - PMF
- Backwater at elev. 1062.6, Tailwater at elev. 1061.0

Table XV summarizes the results of the stability analyses for the required loading conditions.

The sensitivity of the factor of safety to variations in the angle of effective shearing resistance was also assessed for seams V10 and V12 at elevation 1030 m and 1036 m and for three loading conditions as shown on Plate 32.

TABLE XV

FLIP BUCKET STABILITY

LOADING CONDITION	FACTOR OF SAFETY AGAINST SLIDING	REQUIRED FACTOR OF SAFETY
CONSTRUCTION CONDITIONS		
CC-1	1.6	1.3
CC-2	1.3	>1.0
CC-3	1.6	1.3
CC-4	1.2	>1.0
CC-5	1.8	1.3
CC-6	1.2	>1.0
OPERATING CONDITIONS		
OC-1	2.0	1.5
OC-2	1.3	>1.0
OC-3	1.8	1.5
OC-4	1.4	1.3
SEVERE CONDITIONS		
SC-1	1.7	>1.2

Forces Equilibrium Method

The forces equilibrium method was used to calculate the factor of safety against sliding between the structure and the stepped rock foundation. As discussed in Section 3, a modified coefficient of lateral earth pressure of 0.4 was used.

In this analysis an effective angle of shearing resistance of 12.5° was used for seam V12 at elevation 1036 m and an equivalent cross bedding effective angle of shearing resistance of 35° was used for the concrete-excavated rock surface between elevation 1048 m and 1035 m.

This analysis showed that the safety factors for all loading conditions were greater than the required minimum values so that the critical sliding surface will pass entirely through the foundation rock as shown on Plate 31.

4.5.3 Stabilizing Measures

The flip bucket foundation investigation and the stability analyses have shown that the flip bucket will be stable as a mass gravity structure. Anchors will not be required for structure stability and so the galleries together with the grout curtain, drainage provisions, and associated access shafts, ventilation and lighting that were included in the tender design have been deleted from the construction drawings.

4.6 TIE-BACK SUPPORT OF ROCK WALLS

Rock excavations for the abutment monoliths and the flip bucket will result in excavated rock faces of 1 horizontal to 6 vertical slopes at the upstream face of the abutment monoliths on each side of the approach channel and on the upstream face and each side of the flip bucket. A tie-back support system has been designed to provide temporary support for these rock walls during the construction period. Permanent support will be provided by the structures when these are in place.

The tie-back support system consists of simple corrosion protected 36 mm diameter Dywidag single threadbar rock anchors installed in 150 mm diameter drill holes inclined 30° to the horizontal. The bars are anchored in the rock mass with a 6 m grouted bond length located below the base of each rock excavation. Based on an allowable bond strength at the rock/grout interface within the bond length of 100 kPa, a service load capacity of 280 kN has been used for each anchor in the design. Square 800 mm reinforced concrete pads have been designed to distribute the anchor force to the rock face. Local support of the rock face between the anchor locations is provided with minimum 50 mm steel fibre reinforced shotcrete. The pad size and shotcrete thickness will be increased if the rock is found to be more fractured after excavation.

The arrangement of rock anchors in each rock face has been designed to resist a rock loading corresponding to an equivalent lateral pressure coefficient of 0.4 and assumed to vary linearly with depth. Pressure relief drains are provided to prevent significant build-up of water pressure acting on the shotcrete faces.

4.7 SPILLWAY STABILITY

The overall stability of the Spillway structure including the headworks, chute and flip bucket can be assessed by considering potential failure surfaces that pass along the concrete-rock interface and that pass through the bedrock foundation. The design seams that control stability are R7, R1, V18, V16, V12 and V10, which can be considered to be at elevations 1086.5 m, 1068.2 m, 1052.4 m, 1050 m, 1036 m and 1030 m respectively.

Two dimensional analyses were done concentrating on failure surfaces that pass along the design seams. Sliding along the concrete-rock interface is not critical. A simplified stratigraphy was developed from the drillholes and shaft data. The stratigraphy is generally composed of slightly weathered mudrock, intact mudrock and sandstone. In locations where the mudrock is composed of several thin zones of interbedded weathered and intact rocks, it has been categorized as being slightly weathered. Appropriate effective cohesion intercepts were assigned to the shear strength of these zones to reflect the relative amount of weathered and unweathered rock and the proportions of siltstone and claystone in the zones as discussed in Section 3.5.4.2. The simplified stratigraphy together with the cross bedding shear strength parameters are shown on Plate 33.

In the analyses Cases 1 and 3 only, were considered with the pore pressure distributions beneath the ogee and along the sliding planes described in Section 4.3.3 and shown on Plate 18. Cases 2 and 4 are not critical and so were not analyzed in detail.

STABILITY

In the stability analyses the superimposed horizontal water load acting behind the closed gates as the result of the water head difference between the headworks structure and the full supply level was also applied.

The stability analyses were carried out for the following:

- reservoir at full supply level with gates closed, grout curtain and drains effective;
- reservoir at full supply level with gates closed, ineffective grout curtain and drains blocked;
- reservoir at full supply level, gates closed, grout curtain and drains effective, with horizontal seismic loading of 0.065 g.

The results of the analyses carried out for the chute are shown on Plate 33. For all cases the factors of safety are higher than minimum required values. The factors of safety shown on Plate 33 are based on the results of two-dimensional stability analysis. An actual sliding surface would pass through a considerable thickness of rock and soil overburden on both sides of the Spillway because of the deep excavation at the headworks and because the actual sliding surfaces are deep seated. A significant amount of cross bedding shear strength would therefore be mobilized along the edges of the sliding plane, thereby increasing the actual factor of safety.

5.0 STRUCTURAL

5.1 GENERAL

This section describes the criteria that has been used for the design of structural elements of the spillway including the following:

Headworks structure with drainage gallery, approach walls, 2 abutment monoliths, 5 ogee monoliths, bridge, chute wall monoliths, chute and transition slabs, flip bucket and walls, and tie-back retaining walls.

5.2 DESIGN STANDARDS

5.2.1 Codes

The following design codes have been referenced for the structural design of the spillway:

General:	National Building Code of Canada in conjunction with the Alberta Building Regulations.
Concrete:	American Concrete Institute Code, ACI 318-83 "Building Code Requirements for Reinforced Concrete". Canadian Standards Association CSA CAN3-A 23.3-M77.
Structural Steel:	Canadian Standards Association CSA CAN3-S16.1-M84 "Steel Structures for Buildings".

Bridge: Canadian Standards Association CSA CAN3-S6-M78 "Design of Highway Bridges ".

Bridge
Loading: MS-300 Standard Highway Loading.

5.2.2 Standard Practices

Reference has been made to standard practices of the following organizations:

United States Army, Corps of Engineers
United States Department of the Interior, Bureau of Reclamation
Amercian Concrete Institute, Manual of Concrete Practice, 1987.

5.2.3 Material Specifications

The design of the spillway elements described herein has been based on the use of the following material specifications:

Concrete:

Class A1	-	30MPa at 28 days - 20 mm aggregate. Bridge Deck, Walkways and Approach Slabs.
Class B1	-	30 MPa at 28 days - 40 mm aggregate - Final lift ogee slab, transition slab, final lift flip bucket slab.
Class C1	-	30 MPa at 90 days - 40 mm aggregate - Approach Walls, exterior portion of headworks and flip bucket, chute walls, chute slab.

Class D1 - 20 MPa at 90 days - 80 mm aggregate - Interior portion (Mass Concrete) of headworks and flip bucket.

Class E1 - 13.5 MPa at 28 days - 40 mm aggregate - Fill Concrete/Mud slab Concrete

Precast Prestressed Concrete Girders:
- 28 MPa prior to transfer of prestressing forces.
- 35 MPa at 28 days.

Reinforcing Steel:
- CSA G30.12 M77 Grade 400.
- Epoxy coated steel in bridge deck and curbs and at special joints in abutment monoliths.

Concrete Cover to Reinforcing Steel:

Final Ogee Slab and Flip Bucket Slab	- 150 mm
Chute Slab, Chute Walls	- 125 mm
Approach Walls, Piers and Abutments	- 100 mm
Other surfaces not in water passages	- 80 mm

5.2.4 Design Methods

The Limit State Design approach has been used. A load factor of 1.7 has been applied to both live and dead loads. A 25% reduction of the factored load has been allowed for improbable load combinations, such as operating load together with PMF or seismic loading.

Since a limit states analysis approach has been adopted for design of concrete structures, crack sizes were examined in order to limit crack widths to an acceptable level for good serviceability. In particular, crack control has been applied to the water passage faces of the spillway and to locations where prevention of leakage is essential.

5.2.5 Mass Concrete

The following measures have been incorporated to limit heat rise and to control potential cracking of the mass concrete sections in the spillway :

.1 20 MPa specified strength at 90 days - low cement content.
.2 20% minimum and 35% maximum fly-ash by weight of cementitious material.
.3 Type 50 cement.
.4 80 mm coarse aggregate (pumping of concrete with aggregate larger than 20 mm not permitted).
.5 Maximum specified concrete placement temperature of 10°C will require pre-cooling of concrete aggregates, adding of ice to mixing water.
.6 Maximum temperature rise of 22°C.
.7 Curing by water ponding only.
.8 7 day interval between vertical lifts of concrete
.9 1.0 metre maximum concrete lift thicknesses.
.10 Extensive pre-construction trial mix design and testing program including temperature rise test by the contractor have been specified.

5.2.6 Concrete Finishing

The following concrete finishes have been specified for the works:

Finish F1 - Unexposed formed surfaces
Finish F2 - Permanently Exposed formed surfaces
Finish F3 - Exposed Formed surfaces at water passages
- Tolerances - Maximum abrupt irregularity 3mm.
- Maximum gradual irregularity 6 mm in 3 m.

Finish U1 - Unformed Unexposed surfaces - Screeded finish.
Finish U2 - Unformed exposed surfaces - Floated finish.
Finish U3 - Unformed exposed surfaces in water passages
- Trowelled finish tolerances - Maximum abrupt irregularity 3 mm.
- maximum gradual irregularity 6 mm in 3 m.

Finish U4 - Unformed surfaces for pedestrian traffic
Sidewalk finish.
Finish U5 - Bridge Deck Finish.
Finish U6 - Coated Exterior Finish.

The contractor submittals required for finishing include a complete description of methods, materials and equipment with emphasis regarding chute slabs, transition slabs, and final lifts of ogee and flip bucket slabs.

5.2.7 Curing

All concrete will be continuously moist cured as follows:

- Mass Concrete - 14 days water ponding only for horizontal surfaces, continuous moist curing of vertical surfaces

- Chute Walls, Chute Slab - 10 days moist curing
- Bridge Deck, Miscellaneous Concrete - 7 days moist curing

The use of curing compound will not be permitted.

5.2.8 Alkali Aggregate Reactivity

An Alkali Aggregate Reactivity (AAR) Study was carried out for the project. It was concluded that there is a strong potential for AAR if the aggregates at the designated borrow areas are used for the project. Since AAR could lead to premature deterioration of the concrete in the spillway, the following mitigative measures were included in the Contract Documents:

.1 Reactive alkali content of the fly ash was limited to a maximum of 1.4% total equivalent Na_2O.

.2 Reactive alkali content of the cement was specified as maximum 0.6% total equivalent Na_2O.

.3 The total equivalent alkali content of the combined cement and fly ash was limited for each class of concrete to a maximum of $3 kg/m^3$ of concrete.

.4 Total cementitious content of the concrete was minimized consistent with the requirements for durability, strength and performance.

.5 Suitable air entrainment was specified for the concrete.

Additional AAR testing will be carried out as part of the concrete testing program.

5.3 DESIGN APPROACH

5.3.1 Headworks

5.3.1.1 Ogee Monoliths

The typical ogee monolith is shown on Plate 21. It consists of the symmetrical halves of the split pier and the ogee crest structure. Each half of the split pier supports one half of each of the associated bridge span, gate tower structure and lateral water loading acting on the gate as well as the lateral pressure loading from the reservoir water and ice contained within the monolith. The ogee crest structure provides support for the split pier and the gate in the closed position and transmits the loading to the rock foundation.

Each half of the split pier will act as a free-standing cantilever subjected to axial compression and biaxial bending. In the longitudinal direction of the spillway structure, the height to depth ratio of the pier and the magnitude of the lateral loading are such that the bending and shear in this direction are negligible. Structural action of the piers, therefore takes place primarily in the transverse direction (perpendicular to the flow) where the piers are relatively slender and subjected to significant pressure loading from the reservoir water and ice loading as well as pseudo-static seismic inertia forces. For structural design purposes the piers are therefore considered to act as a transversely loaded thin plate subjected to in-plane compression from gravity loading.

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The ogee crest structure is subjected to transverse moments and tensile forces combined with vertical gravity loading from the piers distributed along the joints separating the ogee monoliths in addition to the weight of water contained within the ogee monolith and the distributed base pressure from the rock foundation. Due to the restraining effect of the stiff piers in the longitudinal direction, the ogee crest structure can be expected to behave as a rigid body in this direction except for the upstream and downstream extensions past the pier, which will act as cantilevered slabs subjected to the combined effects of base pressure, uplift pressure and vertical water pressure. In the transverse direction (perpendicular to the flow) the presence of the stiff piers along the edges will cause the ogee base to act as a deep beam on an elastic foundation.

The following load cases, were used as the basis for the design of the pier and the ogee crest structure:

1. Service Load Condition

    - Gate closed
    - Reservoir at FSL
    - Ice load of 150 kN/m
    - Gravity loads due to bridge, gate tower and piers.

2. Ultimate Strength Conditions

    .1 - Gate open
       - Reservoir at PMF
       - Gravity loads due to bridge, gate tower and pier

.2 - Service loads as above
  - Pseudo-static seismic acceleration of 0.065 g, minimum.

Pier Analysis

A finite element method of analysis was used to obtain a realistic assessment of the distribution of vertical and horizontal moments within the pier for each of the load cases considered. The pier was idealized as a thin plate fixed at the base using an assemblage of bending elements combined with boundary springs modelling the fixed boundary conditions at the base. A variable element mesh was used to reflect the variation in the pier geometry and the loading as well as the expected moment gradients. Equivalent nodal loads were used to represent the distribution of transverse loads for each load case. Separate analyses were carried out for each load case to give the distribution of the transverse (perpendicular to the flow) bending moment in the vertical and horizontal direction within the pier and the support reactions at the base of the pier in the form of transverse moments and shear forces. An in-house version of the finite element program SAPIV was used to carry out the analyses.

Ogee Base Analysis

A two dimensional finite element idealization of the ogee crest structure was used to take into account the effect of the elastic rock support and the depth of the ogee section on its behaviour in the transverse direction (perpendicular to the flow). A strip of unit width of the transverse cross-section of the ogee crest structure and the symmetrical half of the pier extending 11.7 m above the top of the ogee base on each side was modelled using two-dimensional quadrilateral and triangular plane strain membrane elements. The location of the cross-section was taken at the location of the maximum edge moment from the pier analysis. The height of the pier included in the model was selected such that the correct moment and shear force for the loading case considered was transferred from the pier to the ogee base when the loading was applied as an equivalent lateral loading to the pier. The rock/structure interaction was simulated using a system of discrete vertical and horizontal springs to represent the vertical stiffness and the lateral friction resistance of the rock respectively. A finite element mesh of variable density quadrilateral and triangular plane strain membrane elements and boundary springs were used to model the symmetrical half of the idealized structure. The analysis was carried out for two loading conditions representing the moment and the shear force associated with the service load combination and the PMF ultimate load combination used in the analysis of the pier.

To examine the sensitivity of the results to variations in the assumed stiffness of the rock each load case was analyzed for an upper and lower bound value of the rock stiffness: A high spring stiffness for both the vertical and the horizontal springs were used to simulate rigid behaviour of the rock as the upper bound value. A vertical spring stiffness based on a subgrade modulus of 30 MPa/m and negligible stiffness for the lateral springs was used to represent the lower bound rock stiffness. The distribution of transverse bending stress over the depth of the ogee base obtained for each load case and simulated rock stiffness was integrated numerically at several sections to give the upper bound and lower bound variations in axial tension and bending moment to be resisted by the ogee base. The variation in ultimate moment and shear in the longitudinal direction of the ogee crest structure acting on the upstream and downstream extensions of the ogee base past the piers was determined for various load cases assuming simple beam cantilever behaviour.

Pier Reinforcement

The variation of vertical and horizontal moments in the pier obtained from the pier analysis for both the service load condition and the ultimate load conditions were used in the design of the pier reinforcement. The serviceability criterion limiting the maximum crack opening width to 0.23 mm governed the design of both the vertical and horizontal reinforcement in critical areas on the water face of the pier, whereas the ultimate load condition represented by the combined seismic load case governed the design of the reinforcement on the joint side of the pier. Shear reinforcement was designed for the upstream tapered pier nosing at the base of the pier to meet the ultimate shear capacity required for the PMF load case.

Ogee Base Reinforcement

Design of the transverse reinforcement in the ogee crest structure was governed by the crack control serviceability criterion and was based on the average of the moments corresponding to the upper and lower bound rock stiffness. Design of the longitudinal reinforcement in the upstream and downstream cantilevered portions of the ogee base was based on ultimate strength considerations. Design of the upstream sections containing the drainage gallery was based on a design procedure applicable for beams with large openings. Longitudinal reinforcement crossing the transverse construction joint at the ogee centerline was designed to have sufficient capacity to transfer across the construction joint the total base shear acting on the upstream portion of the ogee base.

5.3.1.2 Abutment Monoliths - Structural

The structure was designed as an independent reinforced concrete gravity type retaining structure.

To reduce thermal cracking of the mass concrete sections limitations were placed on the concrete mix design parameters, the lift heights, temperature control and joint spacing.

All construction joints are waterstopped to ensure watertightness. Face shells containing reinforcement are of higher strength concrete for improved watertightness and durability.

5.3.1.3 Approach Walls - Structural

The approach walls were designed as semi-gravity type retaining walls on stepped footings on excavated rock or fill concrete.

5.3.1.4 Bridge

General

The bridge will have simple spans corresponding to the seven gates and two additional spans over the abutment monoliths. The deck width is sufficient to carry two lanes of traffic (9.0 m roadway). A walkway minimum 1 m clear width has been incorporated on the downstream side of the bridge superstructure adjacent to the gate hoist structure. Chain link fencing will be installed on the downstream side of the walkway to provide security to the gate tower area.

The bridge has been designed for MS-300 highway loading.

Zero freeboard from the PMF level has been accepted at the bottom of the precast concrete girders.

Materials

The bridge will consist of standard precast, pre-stressed concrete deck bulb-tee girders and a cast-in-place concrete deck, curbs, walkway and approach slabs. Epoxy coated reinforcing steel will be used in all deck concrete. A waterproofing membrane and an asphaltic wearing surface will be provided on the roadway.

Bearings will be of the neoprene pad-type, with stainless steel/teflon contact surfaces at expansion locations. Hollow structural section steel railing will be provided on the curbs. Concrete parapets will provide a transition between bridge railing and roadway flex-beam railing.

Watertight expansion joints will be provided at the centreline of each pier to accommodate temperature and creep movements as well as the anticipated movements due to foundation rebound.

5.3.2 Chute

5.3.2.1 Chute Slab

The chute slopes from the headworks to the beginning of the flip bucket, at a constant slope of approximately 6.4 horizontal to 1 vertical. Under the 600 mm thick chute slab there will be a 100 mm thick concrete mud slab, and 2.4 m of gravel sized (Zone 3) material to ensure drainage of water from the chute and from the rock foundation. A 200 mm sand filter (Zone 2) layer will be provided between the excavated rock surface and the gravel drain material to prevent migration of fine grained particles from the rock into the drain. Perforated polyethylene drain pipes, 300 mm in diameter, will be provided at approximately 30 m intervals in 1 m deep trenches below the excavated rock surface to intercept drainage and seepage. The header drains consist of 600 mm minimum diameter reinforced concrete pipes. A series of manholes have been located along the header drains at each cross drain for maintenance purposes. The total depth of concrete and granular fill overlying the drain pipe system has been set to prevent the pipes from freezing. The slab will be placed in continuous strips which will taper in width from 14 m at the upstream end to 6 m at the downstream end. Longitudinal joints have been provided with shear keys and PVC waterstops. Slab reinforcing was selected to limit long term cracking to approximately 0.5 mm width cracks at a spacing of 3-4 m.

A transition slab has been provided between the headworks and chute slab to accommodate a maximum differential foundation heave of 120 mm.

5.3.2.2 Chute Wall Monoliths

The first three monoliths downstream of the headworks, Monoliths 1, 2 and 3 have been designed as semi-gravity type walls. These walls are founded on fill concrete on excavated rock.

The typical chute wall monoliths (4 to 23) have been designed as cantilever-type walls and are founded on 2.4 m of Zone 3 granular material.

5.3.3 Flip Bucket

The flip bucket was designed essentially as an unreinforced concrete gravity structure. The concrete will be divided into 9 blocks to allow for thermal stress relief in the mass concrete. The construction joints will be reinforced and shear-keyed for the design moments and shears. All joints will be waterstopped and fully grouted to ensure that a continuous structure is obtained.

All exposed faces of the mass concrete sections will receive nominal shrinkage reinforcement. Structural sections including the walls, and the top slab were analysed and reinforced in accordance with the design parameters established for the headworks.

The flip bucket will be provided with 2 heat-traced side wall drains to avoid ice build-up during no flow conditions.

An anchored apron slab will provide erosion protection at the downstream face of the structure.

5.3.4 Miscellaneous Structures

The spillway contract will also include miscellaneous structures as follows:

- Generator building foundation and miscellaneous slabs
- Drainage manholes
- Electrical manholes
- Miscellaneous air vents and electrical shafts

5.3.5 Construction Strategy

There is an uncertainty in the magnitude and time rate expected for the heave/settlement of foundation rock during and after excavation for the spillway headworks structure as discussed in Section 3.9. Therefore an observational approach to construction strategy has been adopted.

Instrumentation monuments have been installed at strategic locations of the structure under other contracts and will be installed prior to the start of excavation and monitored during spillway construction as part of the spillway contract to determine the magnitude and time rate of heave. It is anticipated that, from the unloading-time-displacement relationship of the rebound points placed in drill holes 1-2 m below the ultimate base of the excavation, it will be possible to predict when the major portion of heave will have taken place. This will involve a calibration of the heave model used to estimate the time and magnitude of heave.

The time of installation of secondary concrete for sills and gate guides, placement of the final lift of ogee slab concrete, and placement of the transition slab between the headworks and chute will be dependent upon results of the analysis of the instrumentation monitoring during construction. The final elevation of the ogee crest will be set to account for any anticipated future additional heave that might occur such that the ultimate ogee crest elevation will be 1110.0 m. A lower FSL elevation may need to be specified to accommodate a lower initial ogee crest elevation to avoid additional costs for the gates.

The elevations indicated in this design report are those expected after ultimate heave of the foundation.

5.3.6 Instrumentation

Instruments will be supplied and installed under the Spillway contract including:

- piezometers
- extensometers

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- slope indicators
- standpipes
- frost indicators (chute)
- temperature probes (mass concrete)
- survey reference plates
- strain gauges
- reservoir level indicator

6.0 GATES AND STOPLOGS

6.1 GENERAL

Gates are being supplied and installed under a separate contract from the spillway and detailed criteria are presented in the Gate Design Brief. However, a description of the basic system is provided for reference.

The spillway will be provided with:

- Stoplogs, stoplog dogging devices and a monorail hoist.
- Seven spillway gates complete with cable hoists, heating systems for three gates, towers, hoist bridge, controls, complete with embedded guides and sill beams.

The Spillway Contract includes the supply and installation of embedments in the primary concrete for the stoplog and gate guides and sill beams, as well as anchor bolts for the hoist towers.

6.2 STOPLOGS

One set of stoplogs consisting of 4 sections fabricated in steel has been provided for closing one spillway bay at a time during gate maintenance. The stoplogs will be handled by means of a monorail hoist with a follower beam and will be stored at deck level over the stoplog slots using dogging devices. The stoplogs have been provided with seals and springs to assist in sealing the water passage openings. One of the stoplogs has been provided with a valve for flooding the space between the stoplogs and a closed gate to permit the stoplogs to be raised under balanced conditions. This valve will be actuated by the monorail hoist stoplog follower.

The travel and hoisting functions of the electric monorail hoist will be controlled by pendant push buttons. Devices have been provided for securing the monorail hoist to the structure when it is not in use.

6.3 GATES

The spillway includes seven gates fabricated in steel of the flat, vertical cable hoist lift type with rollers on fixed axles. Each gate is complete with embedded guides, hoist towers, enclosed hoist housing, hoists, position indicating devices, manual and automatic controls. For present purposes, the middle 3 of the 7 gates and their guides have been insulated, heated and provided with bubbler systems. Provision has been made in the guides of the non-heated gates to permit future addition of heaters if required.

The control system is capable of raising or lowering the 7 gates automatically in response to pre-selected reservoir control levels. All 7 gates will operate in a pre-determined sequence through a series of incremental gate openings. The opening (or closing) sequence will continue until the desired reservoir conditions are reached. Manual controls located in the control room and at each spillway gate will allow positioning of the gates at any desired opening.

Instrumentation located in the abutment stairwells at each end of the spillway structure will provide automatic recording of the reservoir level data for input to the gate control system.

Emergency gate operation is provided for by three means:

- Auxiliary diesel electric power supply with automatic transfer switch if grid power fails.
- Auxiliary gas engine drive if both power sources are inoperative.
- The gates are designed to allow closure under self-weight using a fan brake without the use of electrical power.

The gates are sealed on the upstream face by elastomer seals along the sides and bottom edges.

Thermostatically controlled blower heaters of the hot air circulating type are provided for the enclosed insulated gate bodies of the three heated gates. These gates also have wind seals at the sides of the gates. Embedded guides for the heated gates are heated by electric tubular type heaters. Also the sheave blocks of the heated gates will be electrically heated to prevent ice build-up.

6.4 ICE LOADING

The gates and stoplogs have been designed for hydrostatic and wind forces and also for an ice load of 150 kN per lineal metre of width acting over an assumed ice thickness of 600 mm at any depth from the sill to normal reservoir operating level.

6.5 GATE AND STOPLOG SEALS

All gate and stoplog seals are of the oil resistant elastomeric type. All metallic sealing faces are machined stainless steel.

6.6 EMERGENCY GENERATOR

The diesel electric generating set, provided for emergency operation of the spillway gates will be the 800 kW set used elsewhere on site during construction. This set complete with instruments, controls, fuel system, automatic transfer switch, etc. will be relocated near the right bank of spillway.

6.7 HOIST BRIDGE

The spillway hoist bridge consists of a series of rigid steel frames spanning across each bay of the headworks. Each frame is structurally independent of adjacent frames with the vertical joint at each concrete pier continuing through the hoist bridge structure. The hoist housing contains sliding joints which ensure weathertightness as the structure moves in response to heave and/or settlement of the headworks foundation.

The gate guides are extended above the tops of piers to guide the gates up to the fully open and maintenance positions. The upstream side of the guide extensions are removable to permit gate removal from the road bridge.

7.0 MECHANICAL EQUIPMENT, HEATING AND VENTILATION

7.1 GENERAL

This section describes the ventilation and heating requirements and design of systems for: left and right abutment drainage tunnels, spillway headworks drainage gallery and emergency generator building.

7.2 DESIGN CRITERIA

Calculations and equipment selections were based on the following:

- January outside ambient 1% = -33°C, db
- July 3:00 pm outside ambient 2.5% = 31°C, db
- Maximum ambient temperature for electrical equipment operation: 40°C
- Drainage tunnels, gallery ambient temperature: 5°C.

7.3 DESIGN METHODS

7.3.1 Abutment Drainage Tunnels and Spillway Headworks Drainage Gallery

Requirements

Ventilation is required to clear small amounts of possible methane leakage from the surrounding rock formation and groundwater seepage as well as combustion products of gas or diesel powered maintenance equipment up to 10 HP capacity.

Drainage gallery access stairwells will be heated for the prevention of ice build up.

Methods

Ventilation will be provided by 100% outdoor air direct fired air handling units at a rate of 6100 L/s per tunnel and gallery. The ventilation system will normally be off. During personnel visits the system will be turned on to purge the space for one half hour prior to personnel entry and turned off upon exit. An electric gas detection system will also provide "ON/OFF" signals to the ventilation system as required to keep contaminants below acceptable levels. During winter conditions ventilation supply air will be heated to 5°C. During summer conditions air will be supplied at outdoor ambient temperature. Access stairwells will be heated with wall hung electric unit heaters.

7.3.2 Emergency Generator Building

Requirements

Ventilation will be required for cooling during generator operation to keep space temperatures below 40°C.

Heating sufficient to keep space temperatures above 18°C is required.

Methods

Ventilation will be provided by an air handling unit with a capacity of 8500 L/s. Outside air and return air will be mixed for a supply air temperature approximately 10°C below room temperature during cooling cycles.

Heating will be provided by wall hung electric unit heaters.

8.0 ELECTRICAL

8.1 INTRODUCTION

The electrical system in the Spillway Contract consists of power distribution panels, lighting fixtures, power outlets, telephone system, emergency battery units, intrusion alarm system, temperature switches, control cabinets for the Generator Building, Instrument Houses, Drainage Tunnels, Drainage Gallery and the Flip Bucket.

8.2 POWER SUPPLY

Two separate power services will be provided for the Dam to accommodate the Spillway, the Valve Chamber and the various buildings throughout the site. The first service rated at 347/600 volt, 3 phase and 1200 Amp capacity will be located in the Generator Building to feed the Hoist Housing, Instrument House No.1, Right Drainage Tunnel, Flip Bucket, Drainage Gallery, street lighting poles, and the Control Building. The second service rated 347/600 volt, 3 phase and 400 Amp ampacity will be installed in Instrument House No.2 to feed the Left Drainage Tunnel, street lighting poles and the Valve Chamber.

8.3 DISTRIBUTION SYSTEM

Power distribution centres in EEMAC-12 enclosures will be provided in the Generator Building, Flip Bucket, Instrument House No. 2, Hoist Housing and the Valve Chamber. Subpanels will be installed in various locations to feed the light fixtures and the receptacles.

Two electrical systems will be provided in the Drainage Tunnels. The first system consists of watertight and corrosion resistant electrical installations. The second system consists of watertight and explosion proof electrical installation with equipment suitable for Class 1 Division 2 area.

The electrical system in the Drainage Gallery consists of watertight and explosion proof electrical installation with equipment suitable for Class 1 Division 2 area.

General purpose electrical equipment will be provided in the Generator Building and the Instrument Houses. Panels are supplied with EEMAC-12 enclosures.

8.4 INTERIOR LIGHTING

Fluorescent fixtures will be installed in the Instrument Houses and the Generator Building.

Two lighting systems will be installed in the Drainage Tunnels. The first system consists of watertight and corrosion resistant incandescent fixtures. The second system consists of watertight and explosion proof mercury vapour fixtures.

The Drainage Gallery will be illuminated by enclosed, watertight and explosion proof mercury vapour fixtures.

8.5 EXTERIOR LIGHTING

Street lighting poles will be installed along the roadway to provide an acceptable light level. Every luminaire will be provided with a photoelectric cell to monitor the on/off operations. Galvanized steel tapered poles presently installed at the Construction Camp will be relocated and erected along the roadway. Flood lights will be installed to illuminate the headworks downstream of the gates, the flip bucket and downstream of the flip bucket.

8.6 STANDBY POWER

An 800 kw generator set will be installed in the Generator Building to provide full standby power for the 1200 A, 347/600 volt service, should the commercial power fail. This generator will be equipped with an automatic transfer switch.

A 6.5 kw generator set will be installed in Instrument House No. 2 to provide standby power for the control devices and the lighting fixtures. The generator will be equipped with a manual transfer switch.

8.7 TELEPHONE SYSTEM

A main telephone panel will be installed in Instrument House No. 1 and fed from the Ambulance Garage telephone panel. Telephone panels will be provided in the Hoist Housing, Instrument House No. 2, and the Generator Building and will be subfed from the main telephone panel via telephone cables.

Telephone outlets will be provided in the Generator Building, Drainage Tunnels, Instrument Houses and the Drainage Gallery.

8.8 CONVENIENCE OUTLETS

120 VAC convenience outlets will be installed as required. Outlets in the Drainage Tunnels, Drainage Gallery and the Instrument Station will be provided with ground fault protection.

8.9 CONDUITS AND WIRING

Teck cables will be installed to feed main panels, subpanels, transformers, street lighting poles, luminaires and receptacles.

Rigid steel conduits will be used inside the Generator Building and the Instrument Houses sized to the latest edition of the Canadian Electrical Codes.

8.10 GROUNDING

All motors and control equipment will be grounded to mounting bases or panels in a manner approved by the Engineer.

8.11 CONTROLS

Control terminal cabinets will be installed in the Drainage Tunnels and the Drainage Gallery and connected to I/O terminal cabinets located in the Generator Building, Instrument House No. 2, Drainage Gallery and the Hoist Housing. Thermostats, Door Switches, Key Switches, Horns, Selector switches etc. will be connected to the nearest control terminal cabinets via multi conductors teck cables.

A gas detection system will be installed under a separate contract to monitor the gas concentration in the Drainage Tunnels and the Drainage Gallery.

When a predetermined concentration level is reached in any of the above locations, the associated ventilation system will come on and a horn will be activated. In this case the explosion proof lighting luminaires in the Drainage Tunnels could remain on and the remaining incandescent fixtures will be turned off.

8.12 OPERATING VOLTAGES

Electrical equipment will be selected and the controls will be arranged for operation from the following nominal voltages:

Motors	1/3 HP and less	115 volt
Motors	1/2 HP and up	600 volt, 3 PH.
Motor Control Centres		600 volt, 3 PH.
Outlets		120 volt
Control		120 volt
Lighting		120, 347, 600 volt

9.0 CONTROL SYSTEM

9.1 SYSTEM OVERVIEW

The philosophy of the control system design is to provide essential automatic safety functions for the Oldman River Dam project. Primarily, the system will prevent overtopping of the structure during high reservoir level conditions by automatically opening the spillway gates. Precautionary measures will be taken in the design to ensure that the system does not erroneously open the spillway gates and produce unwarranted downstream flooding.

This philosophy is based on the understanding that it is Alberta Environment's intention to station operating staff at the Control Building only during day shift hours on week days, and for the operators to reside off-site, probably in Pincher Creek and be available "on-call" to attend to emergency conditions during their off-hours.

In addition to the dam protection function, the control system will provide functions to assist operating staff during regular operations. Such functions will include:

- Reservoir level monitoring.
- Spillway and valve outflow monitoring.
- Offsite river inflow and outflow monitoring.
- Automatic gate and valve operations through various levels of control.

Control system schematics are illustrated on Plates 34, 35 and 36.

Control and Sensing Equipment

The control system for the project will use established technology, applied in a conservative manner, in deference to the importance of the installation. It will consist of industrial grade transmitters and switches connected to a number of independent industrial computers, referred to as programmable logic controllers (PLC's), located at strategic locations throughout the project site. The PLC's will read the sensor inputs and control the field equipment in accordance to a predefined set of instructions.

Local Supervisory Control Equipment

The PLC's will be linked using dedicated communication lines to an IBM-PC based operator's console for supervisory control and data acquisition functions. The operator's console functions as the process window through which the operator may view the status of the dam system and control it. A second similar computer will be connected to the PLC's and will function as an engineer's console through which the system software maintenance functions will be performed. The engineer's console will also be used a a backup console should the need arise.

Remote Monitoring Equipment

A second IBM-PC based console is presently intended for installation in the Alberta Environment office in Fort MacLeod. This system will be linked to the PLC's at the dam and will allow all monitoring and data logging functions as available on the consoles at the project site, however, no remote control operations will be permitted.

9.2 CONTROL SYSTEM HIERARCHY

The control system is implemented as a series of layers, one overriding the other from a central core outward. The central core control consists of rudimentary manual control devices located in the field adjacent to the equipment being controlled. Outward from the central core control are control overlays consisting of more complex control systems.

The central core is an "Emergency/Normal" keyswitch. As the term infers, this switch is intended to be used in case of emergency or when higher levels of control are not available. It is generally located near the associated device and provides a direct-wired link to the motor starter of the device. Placing the switch in "Emergency" disconnects the device from the PLC and all higher or overriding control layers. The device is directly controlled by the operator using conventional "Raise/Stop/Lower", "Open/Stop/Close", or "Start/Stop" keyswitches. Turning the switch to the "Normal" position passes the control responsibility of the device to the next higher layer.

The first layer contained within the PLC is the equipment "Manual/Automatic" function. The operator may remotely control the equipment from the operator's console by selecting the "Manual" mode and manipulating the device using commands appropriate to it such as "Raise/Stop/Lower", "Start/Stop", or "Open/Stop/Close". Selecting the "Automatic" function passes the control responsibility to the PLC logic block designed to operate the equipment. If the equipment is part of a system, control is passed to the next layer within the PLC logic.

The outermost layer consists of system level "Manual/Automatic" functions within the PLC. A system is defined as consisting of two or more pieces of equipment, each with their individual equipment "Manual/Automatic" functions, working together.

9.3 TUNNEL REGULATING VALVES

Emergency Control

Emergency control of the main regulating valve and two riparian valves of each tunnel will be achieved using switches located in the associated valve chamber. Each valve will be provided with an "Emergency/Normal" and an "Open/Stop/Close" keyswitch. These switches completely disconnect the PLC from the associated valve and are intended only for maintenance purposes or upon complete failure of the control system.

Placing the "Emergency/Normal" switch in the "Emergency" position will disconnect the PLC from the valve and activate the "Open/Stop/Close" switch. The operator will manually position the valve using the key actuated "Open/Stop/Close" switch. A digital indicator mounted next to the keyswitches will provide valve position feedback to the operator.

Valve Position Control

Valve position control is the lowest level of valve control available to the operator through the operator's console and the PLC. It will provide manual positioning of the valves from the Control Building upon failure of the flow control system.

This remote manual control mode will be selected when the local "Emergency/Normal" switch is in the "Normal" position and the valve is selected for "Manual" operation on the console in the Control Building. The operator will use the console to input the desired valve position (valve setpoint) to the PLC. The operator then issues a "Proceed" command to initiate the change. The valve position control algorithm in the PLC will compare the desired position with the current valve position and move the valve in the appropriate direction until the desired position is achieved. No further positioning of the valve is done unless operator initiated. The operator will have both valve setpoint and position displayed on the console screen.

Tunnel Flow Control

The flow in each tunnel will be measured using two main systems. The primary measurement system is an ultrasonic flowmeter located in the upstream transition. The secondary flow measurement will be by calibrated gate openings. The pressure differential between the reservoir head measured at the tunnel inlet and a pressure top downstream of the main guard valve will also provide flow rate data.

Flow control will be provided as the normal operating mode for the tunnel valves. Valve position control or Emergency control should only be necessary if the flow control system is not functioning.

Flow control is selected when the local "Emergency/Normal" switch for the valve is in the "Normal" position and the valve is selected for "Automatic" operation on the console in the Control Building. The operator will use the console to input the desired tunnel flow rate (flow setpoint) to the PLC. The operator then issues a "Proceed" command to initiate the change. A flow control algorithm in the PLC will compare the flowrate from the ultrasonic flowmeter with the desired flow setpoint. The algorithm will make changes to the valve setpoints until the flow is at setpoint. Once at setpoint, no further positioning of the valve will occur unless operator initiated. Flow, flow setpoint, valve positions, and valve setpoints will be displayed on the console screen.

9.4 TUNNEL GUARD VALVES

The guard valves are configured in a supervised manner, to close automatically should the valve chamber flood or the main regulating valve fail resulting in a high flow. The means of sensing flow will be independent from the ultrasonic flowmeter devices. The guard valves may also be manually operated using an "Open/Stop/Close" keyswitch in the valve chamber. No remote operation from the Control Building will be provided.

Opening of the valve will be inhibited if the differential across the valve is too high as sensed by a differential pressure switch. Valve position indication will be via a mechanical indicator located on the end of the operating shaft. End travel limit switches will provide remote indication of the valve status on the operator's console.

9.5 SPILLWAY GATES

Emergency Control

Emergency control of the spillway gates will be provided locally in the hoist housing. The control will consist of "Emergency/Normal" and "Raise/Stop/Lower" keyswitches located in each gate control panel. In addition each panel will have a digital indicator for the gate to provide gate position feedback. These emergency control switches completely disconnect the PLC from the gates and are intended only for maintenance purposes or upon complete failure of the control system.

Local emergency gate control will be achieved by placing the "Emergency/Normal" keyswitch to the hoist housing in the "Emergency" position. This will disconnect the gate from the PLC located in the Control Building. The operator will manually position the gate in question using the "Raise/Stop/Lower" keyswitch located on the gate control panel or by plugging in a portable pendant station in a receptacle located near the bridge deck walkway.

Gate Position Control

Gate position control is the lowest level of gate control available to the operator through the operator's console and the PLC. It will provide manual positioning of the gates from the Control Building upon failure of the flow control system.

This remote manual control mode will be selected when the local "Emergency/Normal" switch is in the "Normal" position and the gate has been selected for "Manual" operation on the console in the Control Building. The operator will use the console to input the desired gate position (gate setpoint) to the PLC. The operator then issues a "Proceed" command to initiate the change. A gate position control algorithm in the PLC will compare the current gate position with the gate setpoint and move the gate into the appropriate direction until setpoint is achieved. Once at setpoint no further positioning of the gates will occur unless operator initiated. The operator will have gate setpoint, gate position, and flow through the gate displayed on the console screen.

Spillway Flow Control

The spillway flowrate is computed by using the head on the gates and the gate opening in a gate flow equation. The spillway flow is the sum of the individual gate flow rates.

Spillway flow control will be provided as the normal operating procedure for the spillway gates. It will be selected when the local "Emergency/Normal" keyswitch is in the "Normal" position and the gate has been selected for "Automatic" operation on the operator's console. The operator will use the console to input the desired spillway flowrate (flow setpoint) to the PLC. The operator then issues a "Proceed" command to initiate the change. A flow control algorithm in the PLC will use the head on the gates and the flow setpoint to calculate the desired gate setpoints for the gates. The gate controllers in the PLC will move the gates to the gate setpoints. Once at setpoint, no further positioning of the gates will occur unless operator initiated. All pertinent data related to the control function including setpoints, flow rates and gate positions will be displayed on the console screen.

Automatic Dam Protection

The automatic dam protection function is included to prevent overtopping of the dam. It is only expected to be used as a last resort should operations staff fail to keep the reservoir level at a safe elevation.

The dam protection function will override all other forms of control excepting Emergency operation. In other words, it will be enabled as long as the "Emergency/Normal" switches for the gates are in the "Normal" position.

A level control algorithm in the PLC will automatically start raising the spillway gates when the reservoir exceeds a specified elevation. The algorithm will continue to raise the gates as the level rises, and lower them when the level falls. The event will be considered an alarm condition, and the control system will first generate alarms before the function occurs, as well as while it is occurring. The "alarming" and "gate raising" algorithms will be designed to provide as much warning as possible to operating staff while maintaining the integrity of the structure.

9.6 RESERVOIR LEVEL MEASUREMENT

Spillway Forebay Level

Level transmitters installed in stilling wells on either side of the spillway will be used to provide accurate and redundant level measurement of the level in front of the spillway. They will measure the level from the sill of the gates to top of dam. In addition, float actuated switches located in the dam stilling wells will incorporate a graduated sight glass for local indication and verification of the level signal.

Reservoir Level

Two sensing pipes running from the inlet of the diversion tunnels to the valve chambers will provide pressure signals for the entire range of the reservoir. Level transmitters on the sensing pipes will be connected to the valve chamber PLC for access by the operator's console in the Control Building. Pressure indicators on the sensing pipes will provide local indication and verification of the level signals.

9.7 RIVER MONITORING

Upstream Level Monitoring

The levels of the three rivers upstream of the reservoir will be measured and telemetered back to the PLC at the reservoir site. The operating console computer will use the river level data to calculate and display the flow in each stream as well as sum the individual flows.

Downstream Level Monitoring

The level of the river downstream of the reservoir will be measured and telemetered back to the PLC. The operating console computer will use the level to calculate and display the river flow.

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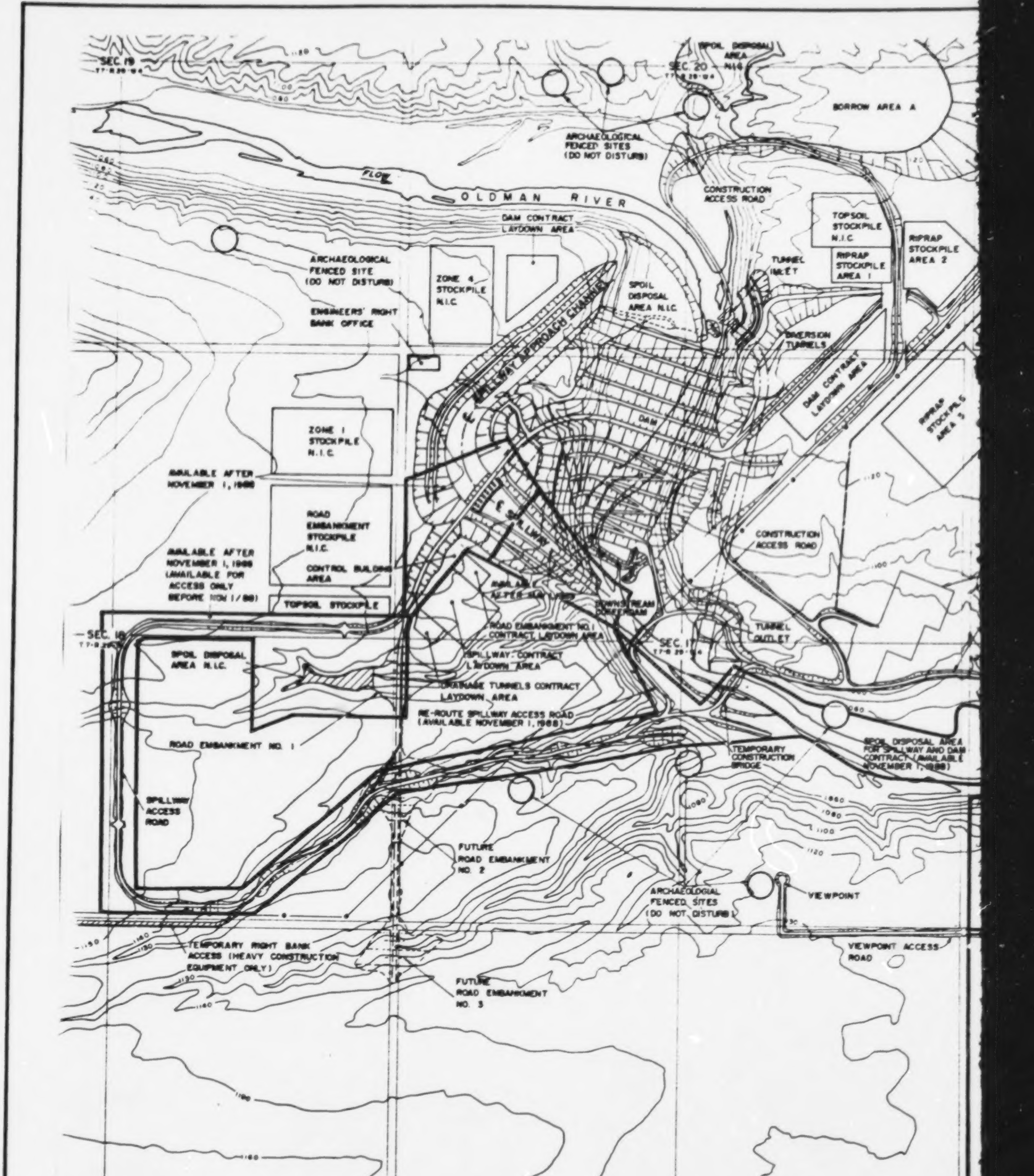
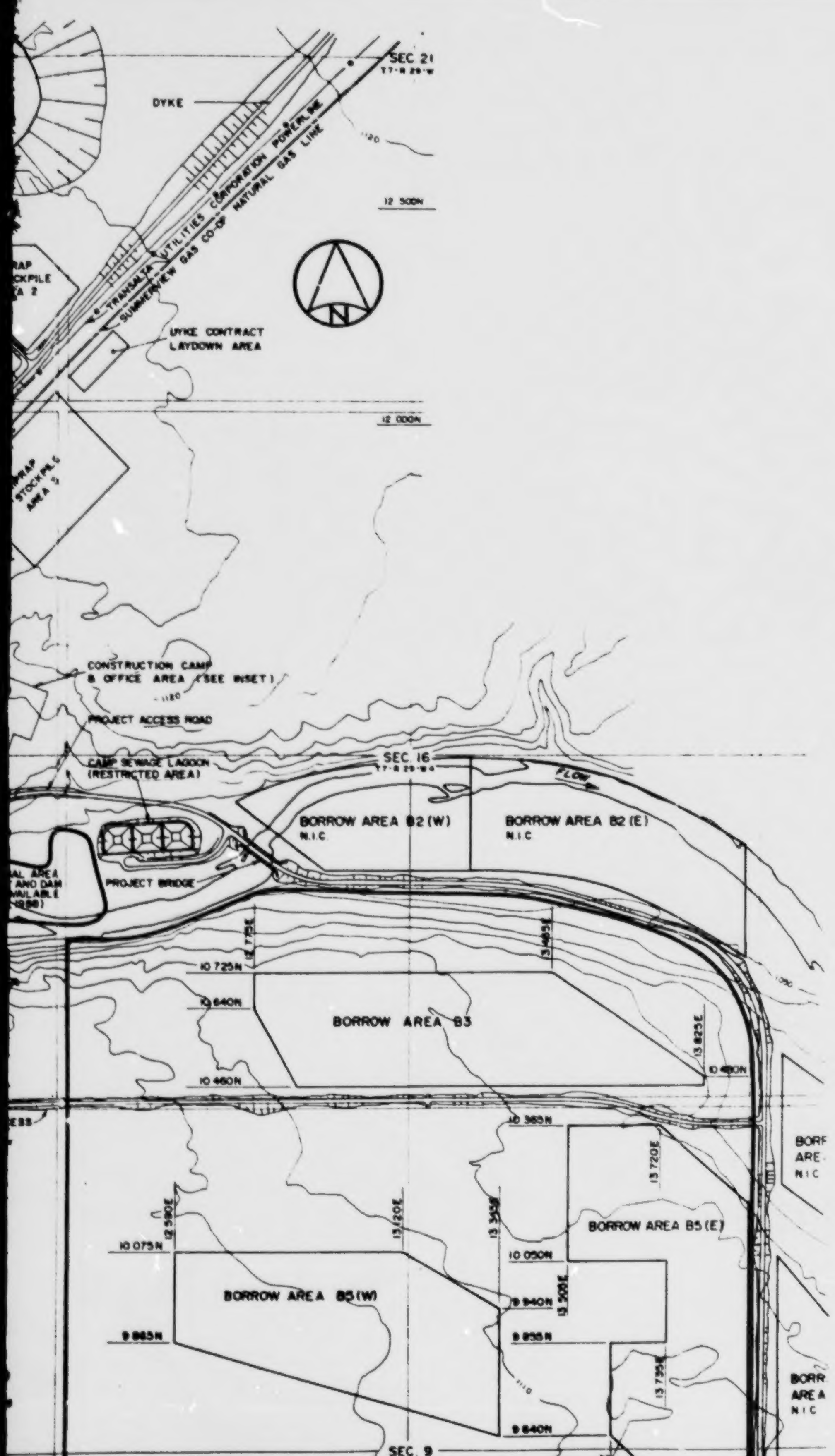
APPENDIX

1


SEC. 19
T7-R29-W4
SEC. 20
T7-R29-W4
SPOIL DISPOSAL AREA N.I.C.
BORROW AREA A
ARCHAEOLOGICAL FENCED SITES (DO NOT DISTURB)
FLOW
OLDMAN RIVER
CONSTRUCTION ACCESS ROAD
DAM CONTRACT LAYDOWN AREA
TOPSOIL STOCKPILE N.I.C.
RIPRAP STOCKPILE AREA 2
RIPRAP STOCKPILE AREA 1
ARCHAEOLOGICAL FENCED SITE (DO NOT DISTURB)
ZONE 4 STOCKPILE N.I.C.
SPOIL DISPOSAL AREA N.I.C.
TUNNEL INLET
ENGINEERS' RIGHT BANK OFFICE
SPILLWAY APPROACH CHANNEL
DIVERSION TUNNELS
DAM CONTRACT LAYDOWN AREA
DAM
RIPRAP STOCKPILE AREA 3
ZONE 1 STOCKPILE N.I.C.
AVAILABLE AFTER NOVEMBER 1, 1988
℄ SPILLWAY
ROAD EMBANKMENT STOCKPILE N.I.C.
CONSTRUCTION ACCESS ROAD
AVAILABLE AFTER NOVEMBER 1, 1988 (AVAILABLE FOR ACCESS ONLY BEFORE NOV 1/88)
CONTROL BUILDING AREA
AVAILABLE AFTER MAY 1, 1989
TOPSOIL STOCKPILE
DOWNSTREAM COFFERDAM
SEC. 18
T7-R29-W4
ROAD EMBANKMENT NO. 1 CONTRACT LAYDOWN AREA
TUNNEL OUTLET
SEC. 17
T7-R29-W4
SPOIL DISPOSAL AREA N.I.C.
SPILLWAY CONTRACT LAYDOWN AREA
DRAINAGE TUNNELS CONTRACT LAYDOWN AREA
RE-ROUTE SPILLWAY ACCESS ROAD (AVAILABLE NOVEMBER 1, 1988)
ROAD EMBANKMENT NO. 1
SPOIL DISPOSAL AREA FOR SPILLWAY AND DAM CONTRACT (AVAILABLE NOVEMBER 1, 1988)
TEMPORARY CONSTRUCTION BRIDGE
SPILLWAY ACCESS ROAD
FUTURE ROAD EMBANKMENT NO. 2
ARCHAEOLOGIAL FENCED SITES (DO NOT DISTURB)
VIEWPOINT
TEMPORARY RIGHT BANK ACCESS (HEAVY CONSTRUCTION EQUIPMENT ONLY)
VIEWPOINT ACCESS ROAD
FUTURE ROAD EMBANKMENT NO. 3
1180
1160
1120
1100
1080
1060
SEC. 7
T7-R29-W4
SEC. 8
T7-R29-W4

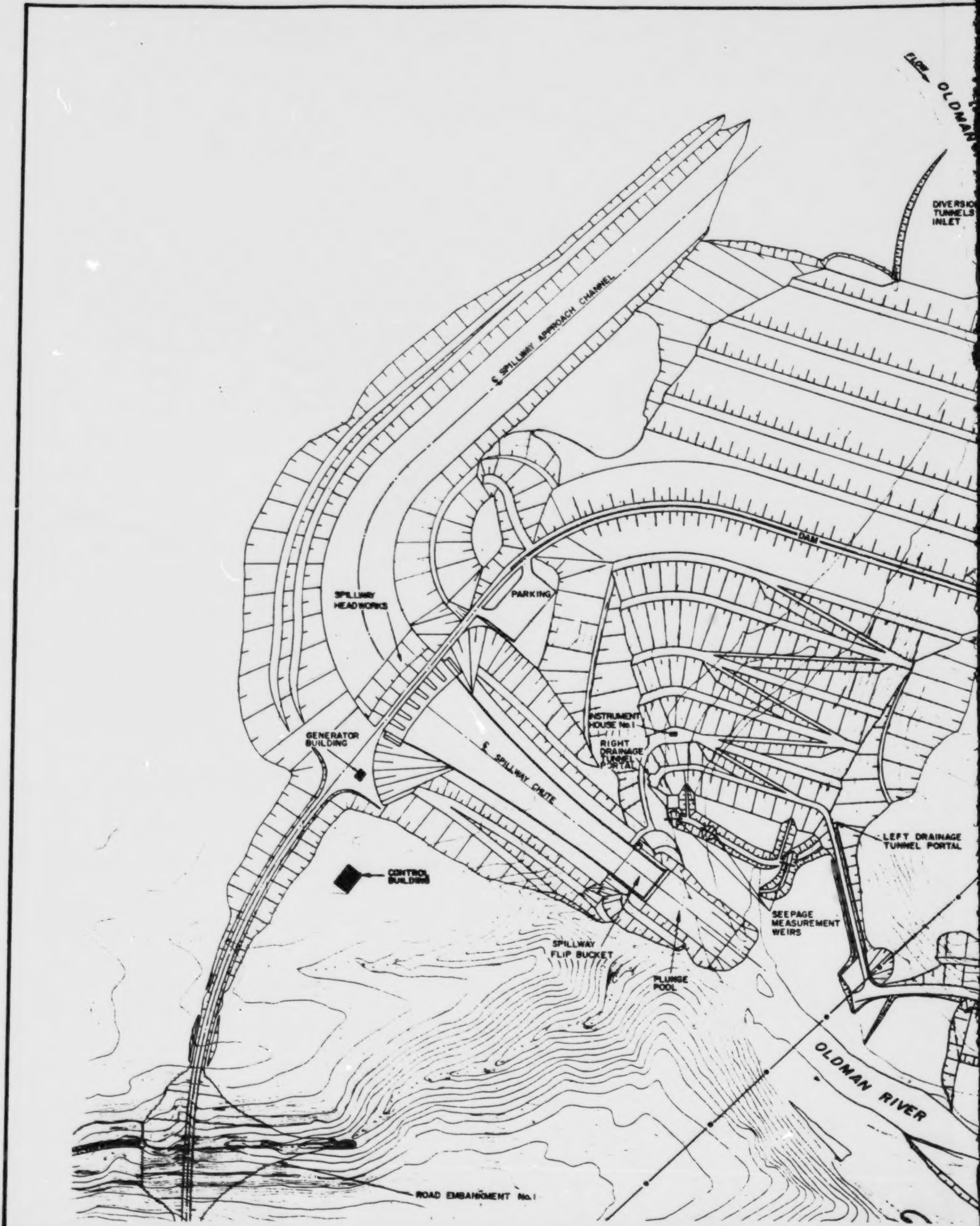

SEC. 21
T7-R 29-W
DYKE
TRANSALTA UTILITIES CORPORATION POWERLINE
SUMMERVIEW GAS CO-OP NATURAL GAS LINE
1120
12 500N
N
RAP
CKPILE
A 2
DYKE CONTRACT
LAYDOWN AREA
12 000N
RAP
STOCKPILE
AREA 3
CONSTRUCTION CAMP
& OFFICE AREA (SEE INSET)
1120
PROJECT ACCESS ROAD
CAMP SEWAGE LAGOON
(RESTRICTED AREA)
SEC. 16
T7-R 29-W4
FLOW
BORROW AREA B2 (W)
N.I.C.
BORROW AREA B2 (E)
N.I.C.
PROJECT BRIDGE
AL AREA
AND DAM
VAILABLE
1988)
12 775E
13 465E
10 725N
10 640N
BORROW AREA B3
1090
13 825E
10 480N
10 460N
ESS
10 365N
13 720E
BORR
ARE
N.I.C
12 590E
13 120E
13 345E
BORROW AREA B5 (E)
10 075N
10 050N
BORROW AREA B5 (W)
9 940N
13 200E
9 863N
9 935N
1110
13 735E
BORR
AREA
N.I.C
9 840N
SEC. 9
T7-R 29-W4
00N
05E

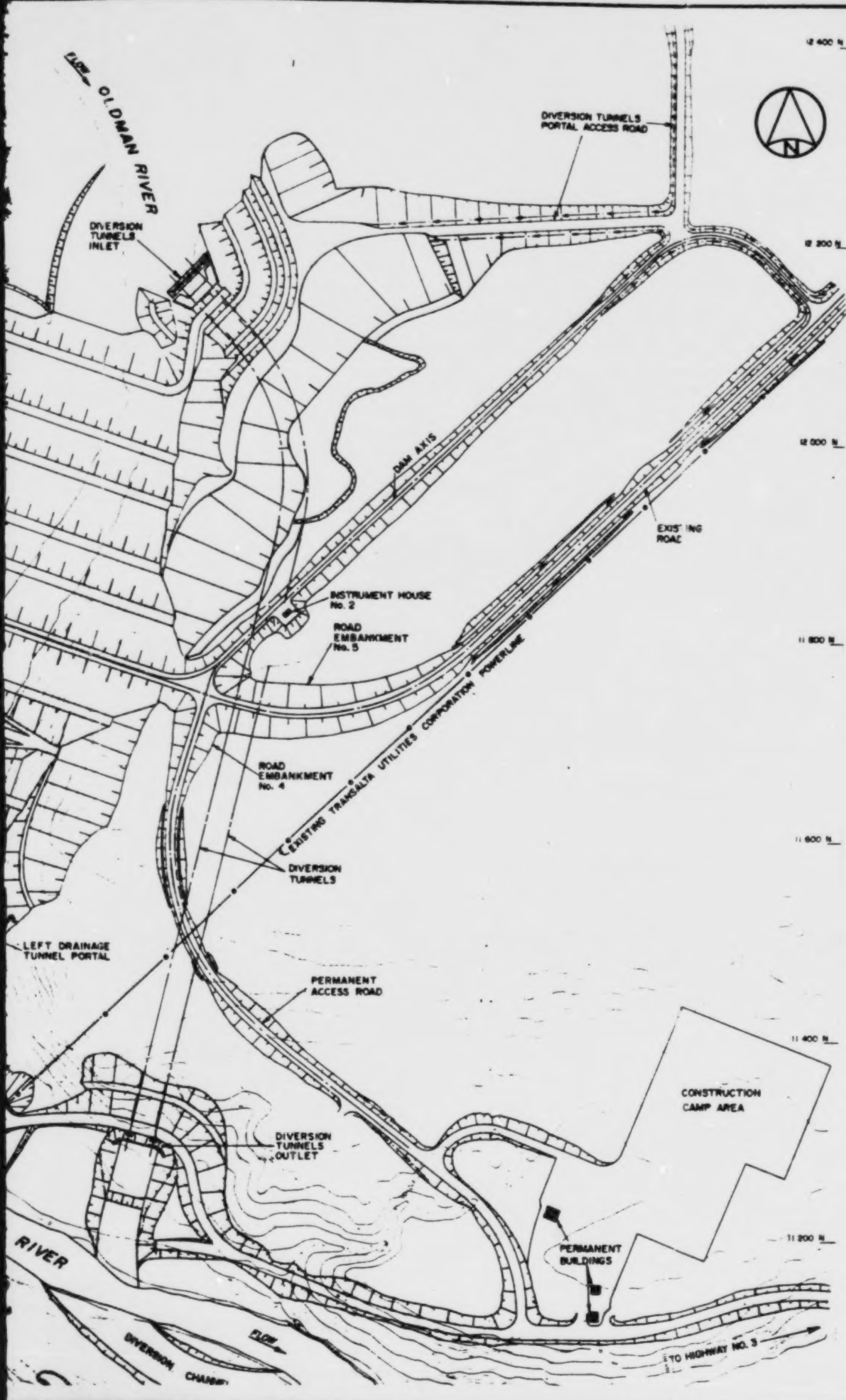
Alberta

OLDMAN RIVER DAM

PROJECT SITE

PLATE 1


FLOW
OLDMAN
DIVERSIO
TUNNELS
INLET
℄ SPILLWAY APPROACH CHANNEL
DAM
SPILLWAY
HEADWORKS
PARKING
INSTRUMENT
HOUSE No. I
RIGHT
DRAINAGE
TUNNEL
PORTAL
GENERATOR
BUILDING
℄ SPILLWAY CHUTE
LEFT DRAINAGE
TUNNEL PORTAL
CONTROL
BUILDING
SEEPAGE
MEASUREMENT
WEIRS
SPILLWAY
FLIP BUCKET
PLUNGE
POOL
OLDMAN RIVER
ROAD EMBANKMENT No. I

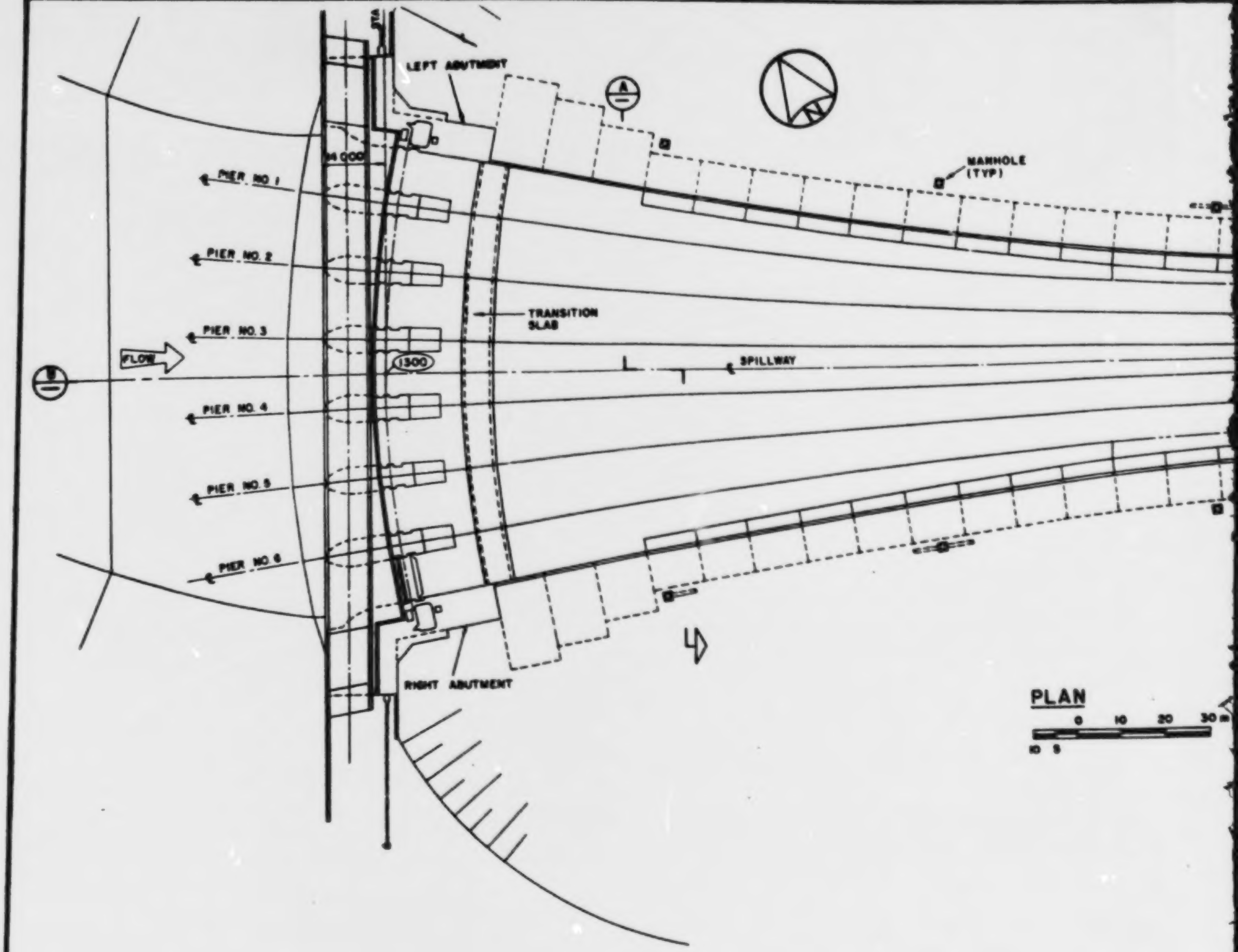

FLOW
OLDMAN RIVER
DIVERSION TUNNELS INLET
DIVERSION TUNNELS PORTAL ACCESS ROAD
12 400 N
12 200 N
12 000 N
11 800 N
11 600 N
11 400 N
11 200 N
N
DAM AXIS
EXISTING ROAD
INSTRUMENT HOUSE No. 2
ROAD EMBANKMENT No. 5
ROAD EMBANKMENT No. 4
EXISTING TRANSALTA UTILITIES CORPORATION POWERLINE
DIVERSION TUNNELS
LEFT DRAINAGE TUNNEL PORTAL
PERMANENT ACCESS ROAD
CONSTRUCTION CAMP AREA
DIVERSION TUNNELS OUTLET
PERMANENT BUILDINGS
RIVER
FLOW
DIVERSION CHANNEL
TO HIGHWAY NO. 3

Alberta

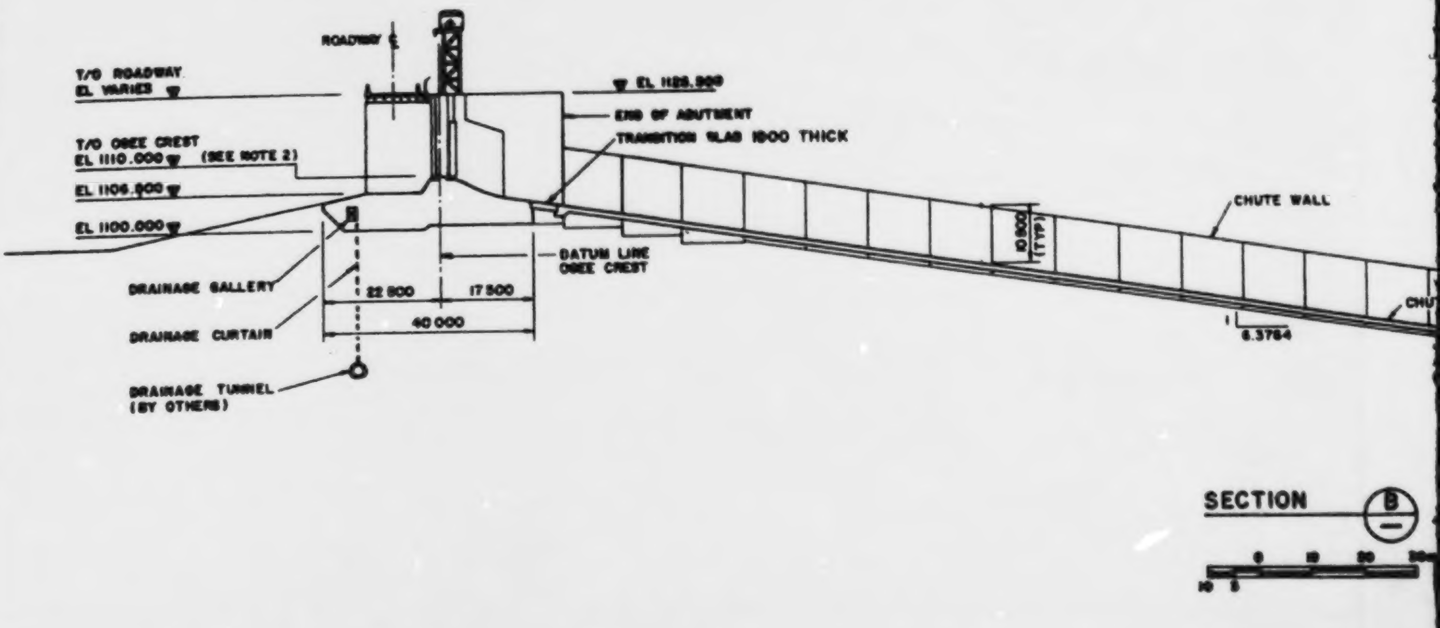
OLDMAN RIVER DAM

PROJECT LAYOUT

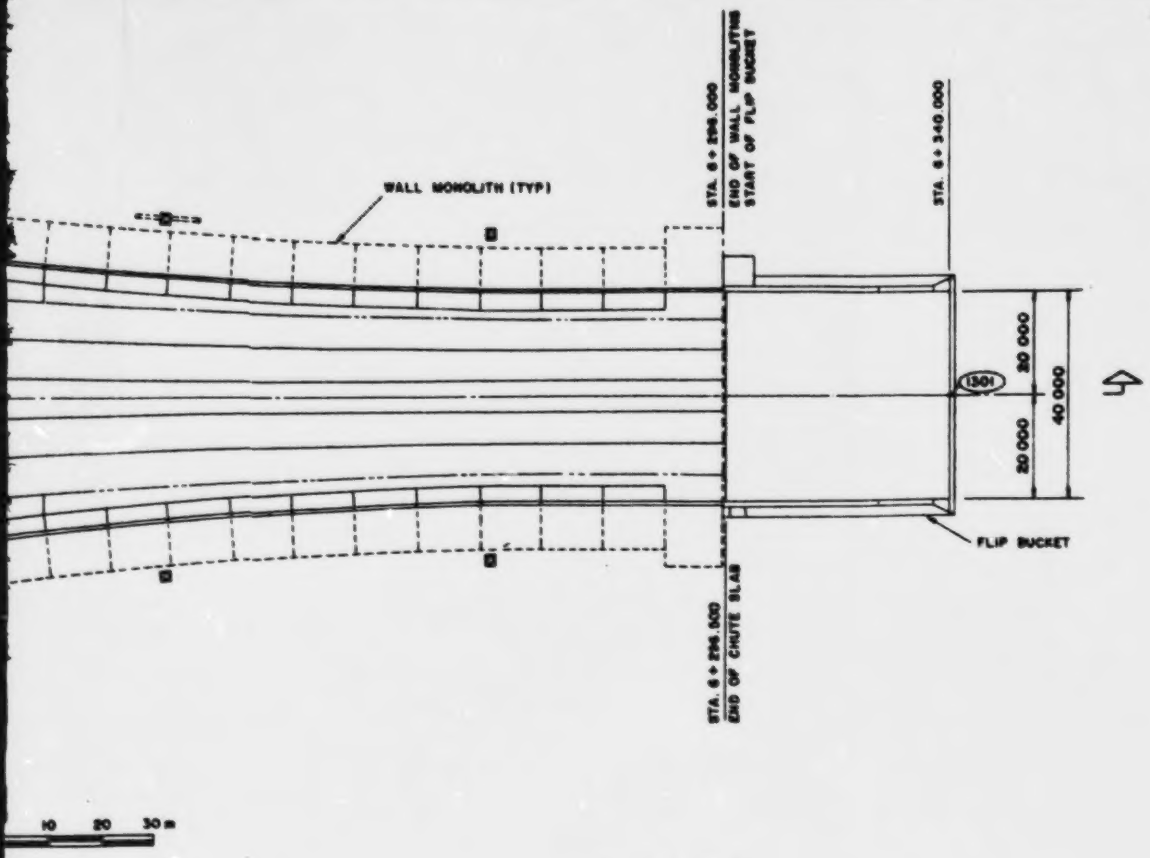
PLATE 2

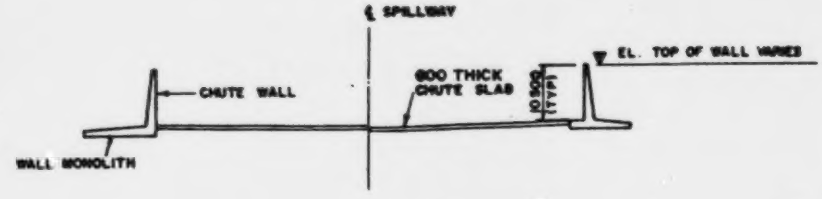

LEFT ABUTMENT
A
MANHOLE (TYP)
34.000
℄ PIER NO 1
℄ PIER NO. 2
℄ PIER NO. 3
℄ PIER NO. 4
℄ PIER NO. 5
℄ PIER NO. 6
TRANSITION SLAB
FLOW
B
1300
℄ SPILLWAY
RIGHT ABUTMENT
0
10
20
30 m
10
5

PLAN


ROADWAY ℄
T/O ROADWAY EL VARIES
EL 1125.500
END OF ABUTMENT
TRANSITION SLAB 1000 THICK
T/O OGEE CREST EL 1110.000 (SEE NOTE 2)
EL 1106.500
EL 1100.000
CHUTE WALL
10.500 (TYP)
DATUM LINE OGEE CREST
DRAINAGE GALLERY
22.500
17.500
40.000
DRAINAGE CURTAIN
DRAINAGE TUNNEL (BY OTHERS)
1
6.3764
CHU
0
10
20
30
10
5

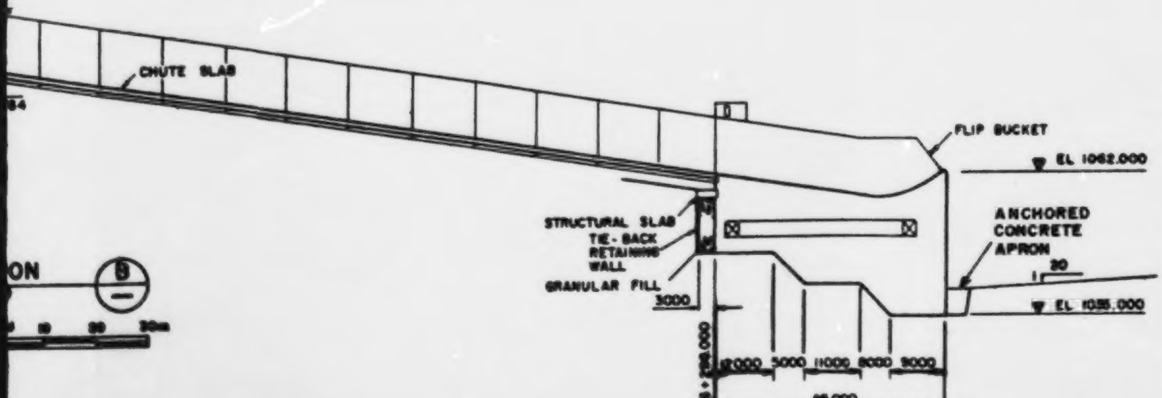
SECTION B


WALL MONOLITH (TYP)
STA. 6 + 296.000
END OF WALL MONOLITHS
START OF FLIP BUCKET
STA. 6 + 340.000
1301
20 000
20 000
40 000
FLIP BUCKET
STA. 6 + 296.000
END OF CHUTE SLAB
10
20
30 m


℄ SPILLWAY
EL. TOP OF WALL VARIES
CHUTE WALL
600 THICK
CHUTE SLAB
10 500 (TYP)
WALL MONOLITH

SECTION Ⓐ

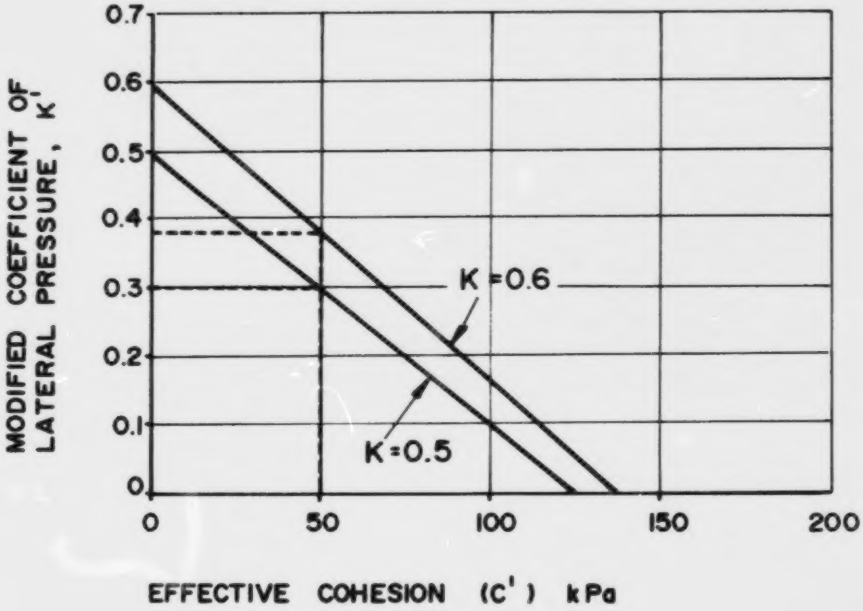
0
10
20
30 m
10
5


TE WALL
CHUTE SLAB
84
FLIP BUCKET
EL 1062.000
STRUCTURAL SLAB
TIE-BACK
RETAINING
WALL
GRANULAR FILL
ANCHORED
CONCRETE
APRON
1
20
3000
EL 1035.000
STA. 6 + 296.000
12000
5000
11000
8000
9000
45 000
ON
B
0
10
20
30 m

Alberta
OLDMAN RIVER DAM

GENERAL ARRANGEMENT

PLATE 3



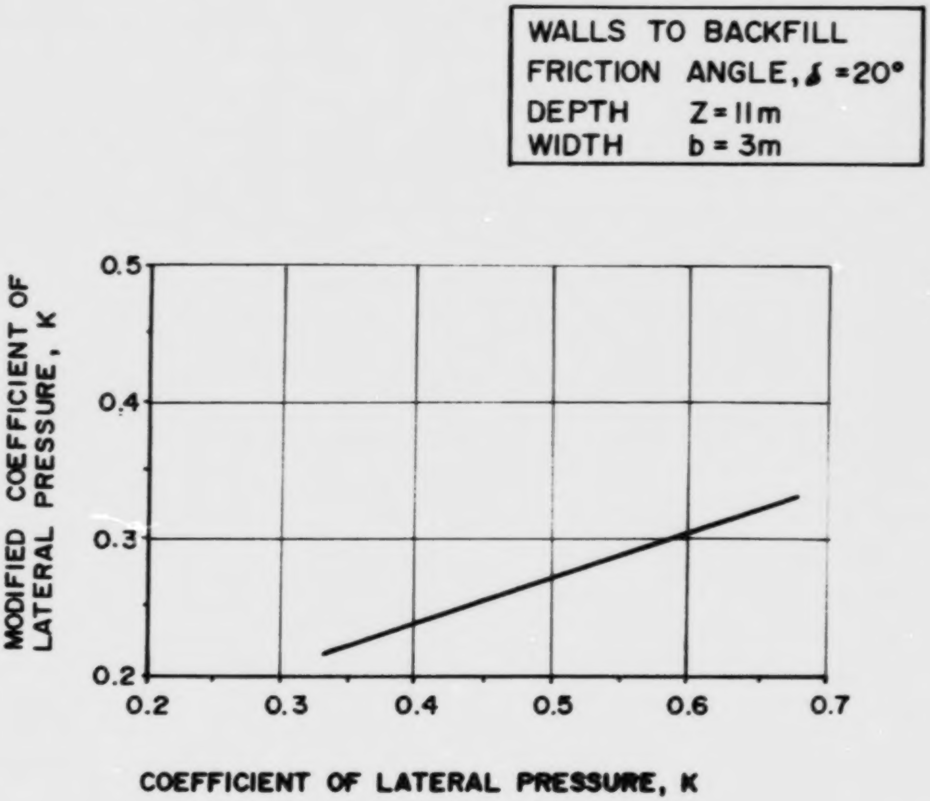
EFFECT OF COHESION ON LATERAL PRESSURE

Alberta

OLDMAN RIVER DAM

EFFECT OF COHESION
ON LATERAL PRESSURE

PLATE 4

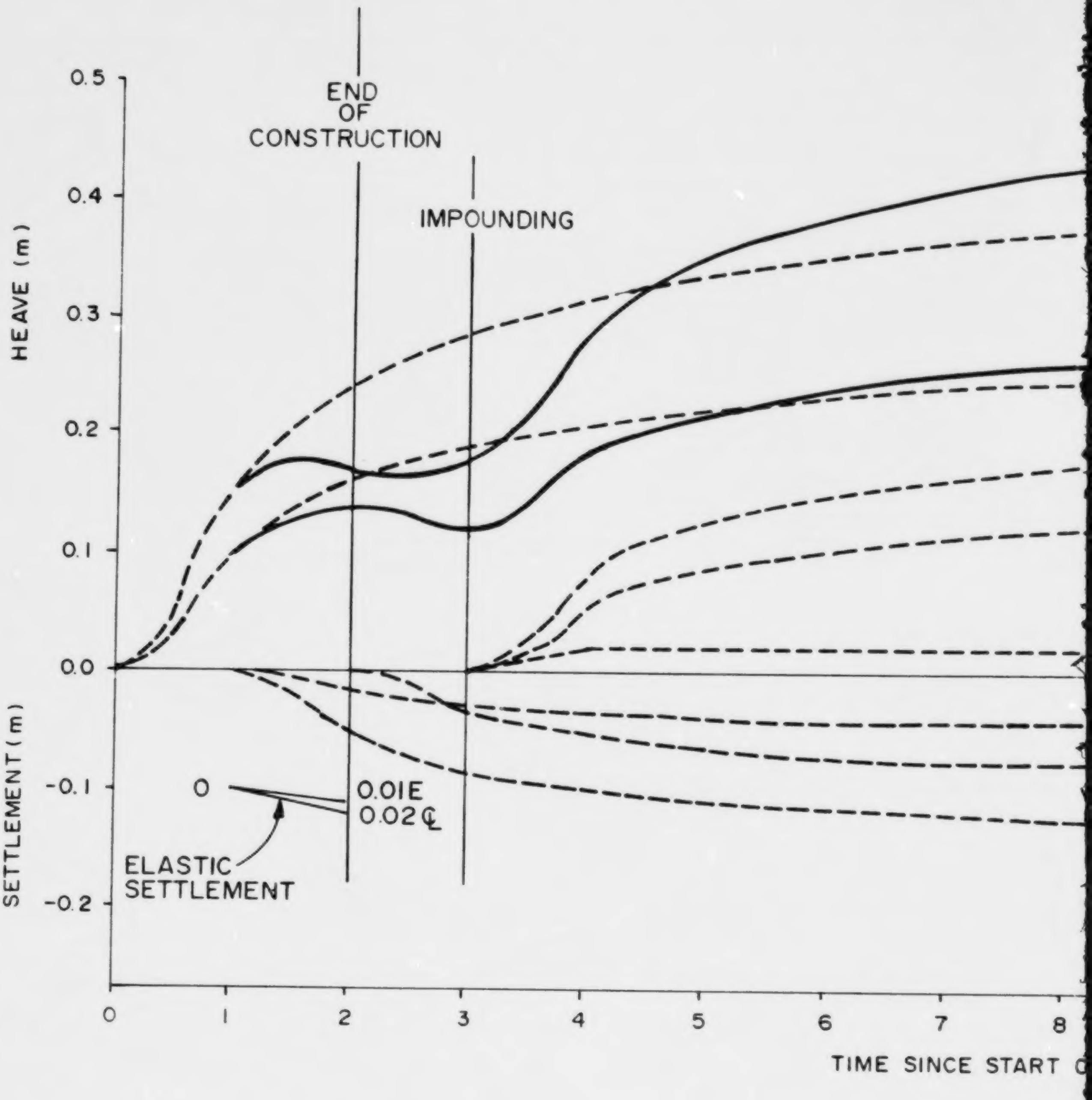


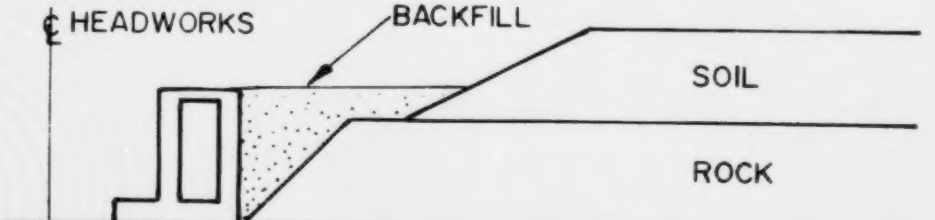
EFFECT OF ARCHING ON LATERAL PRESSURE

Alberta
OLDMAN RIVER DAM

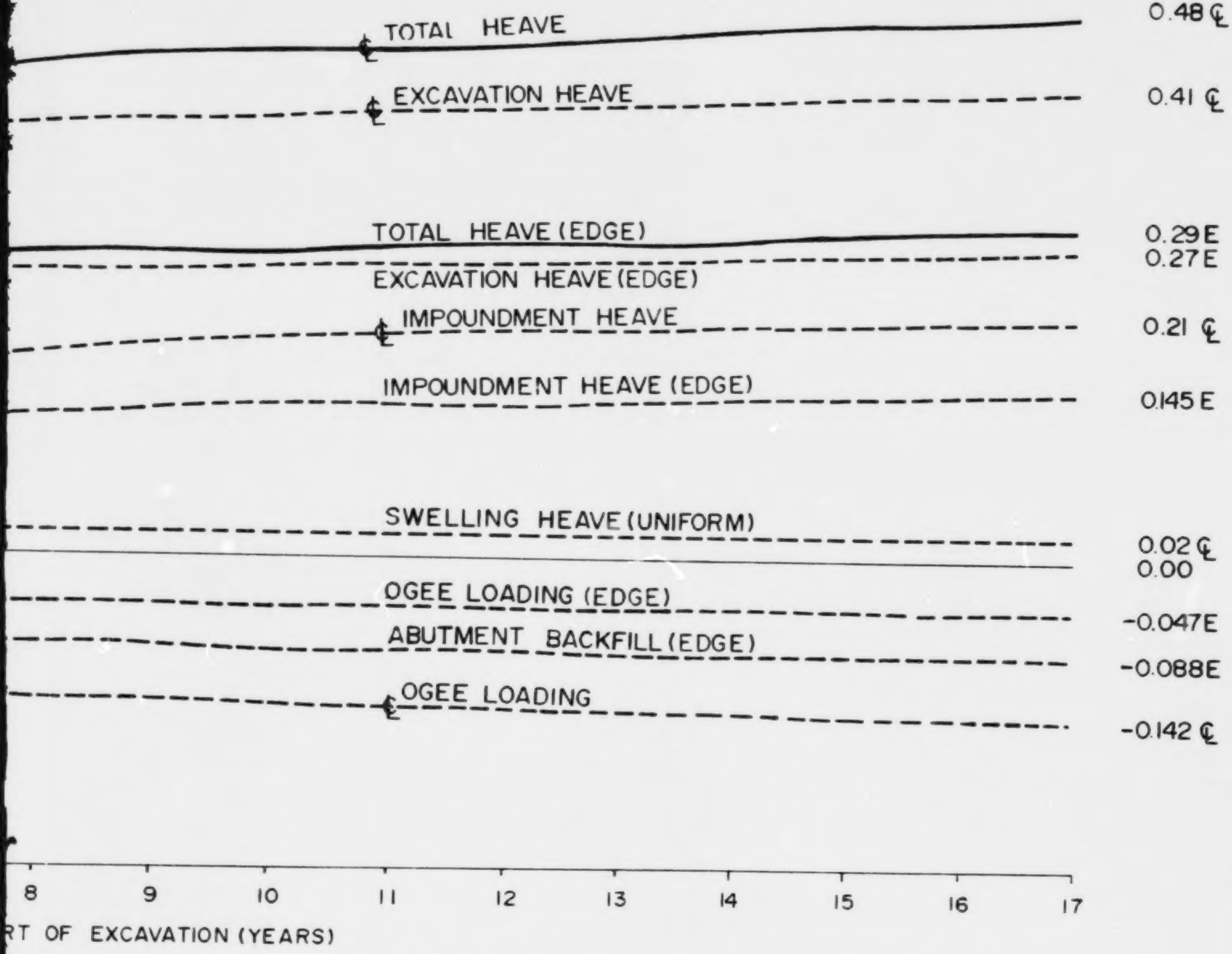
EFFECT OF ARCHING ON LATERAL PRESSURE

PLATE 5




℄ HEADWORKS
BACKFILL
SOIL
ROCK
EDGE OF EXCAVATION

TYPICAL HEADWORKS SECTION

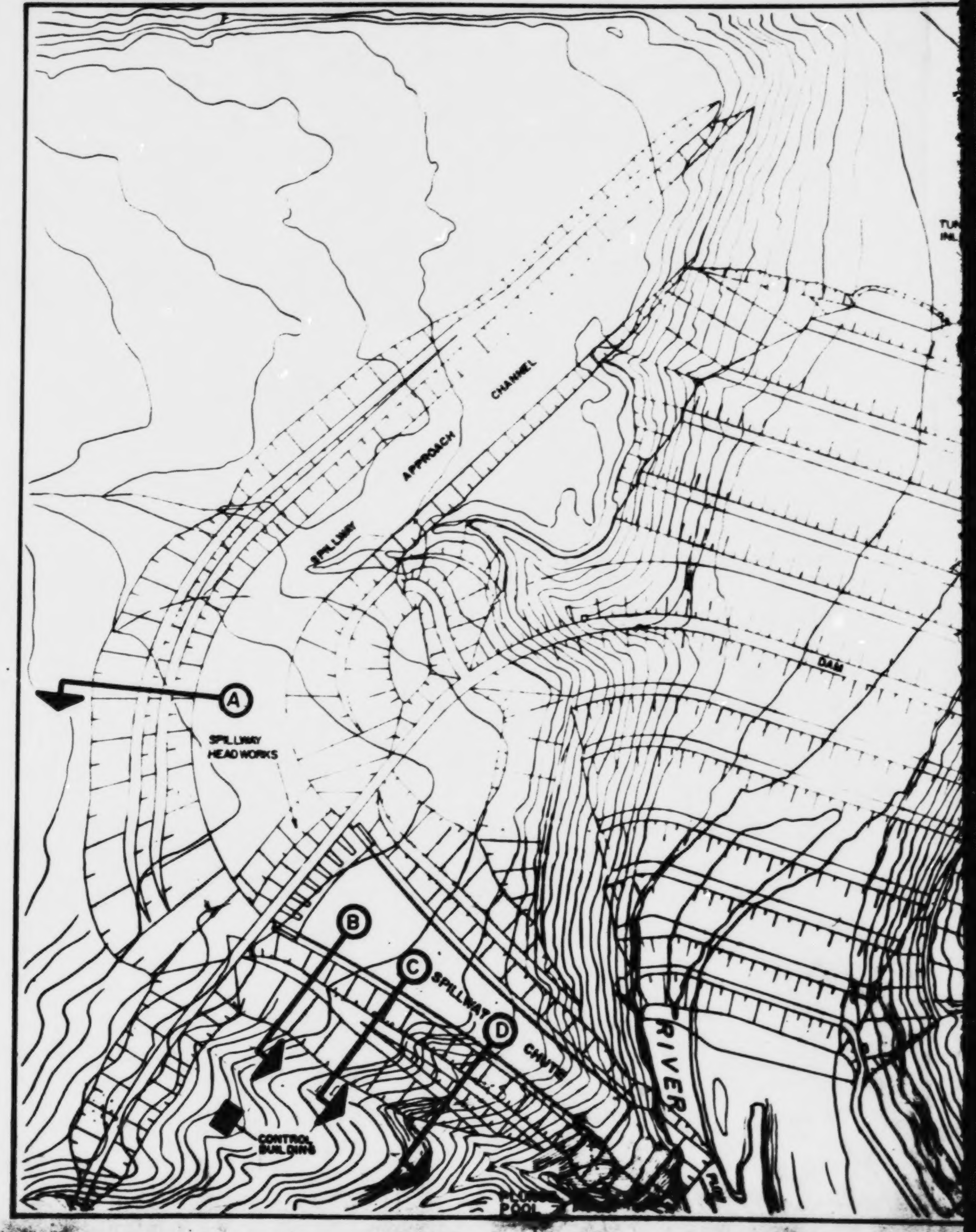


MAXIMUM ESTIMATED DIFFERENTIAL HEAVE 0.14 m

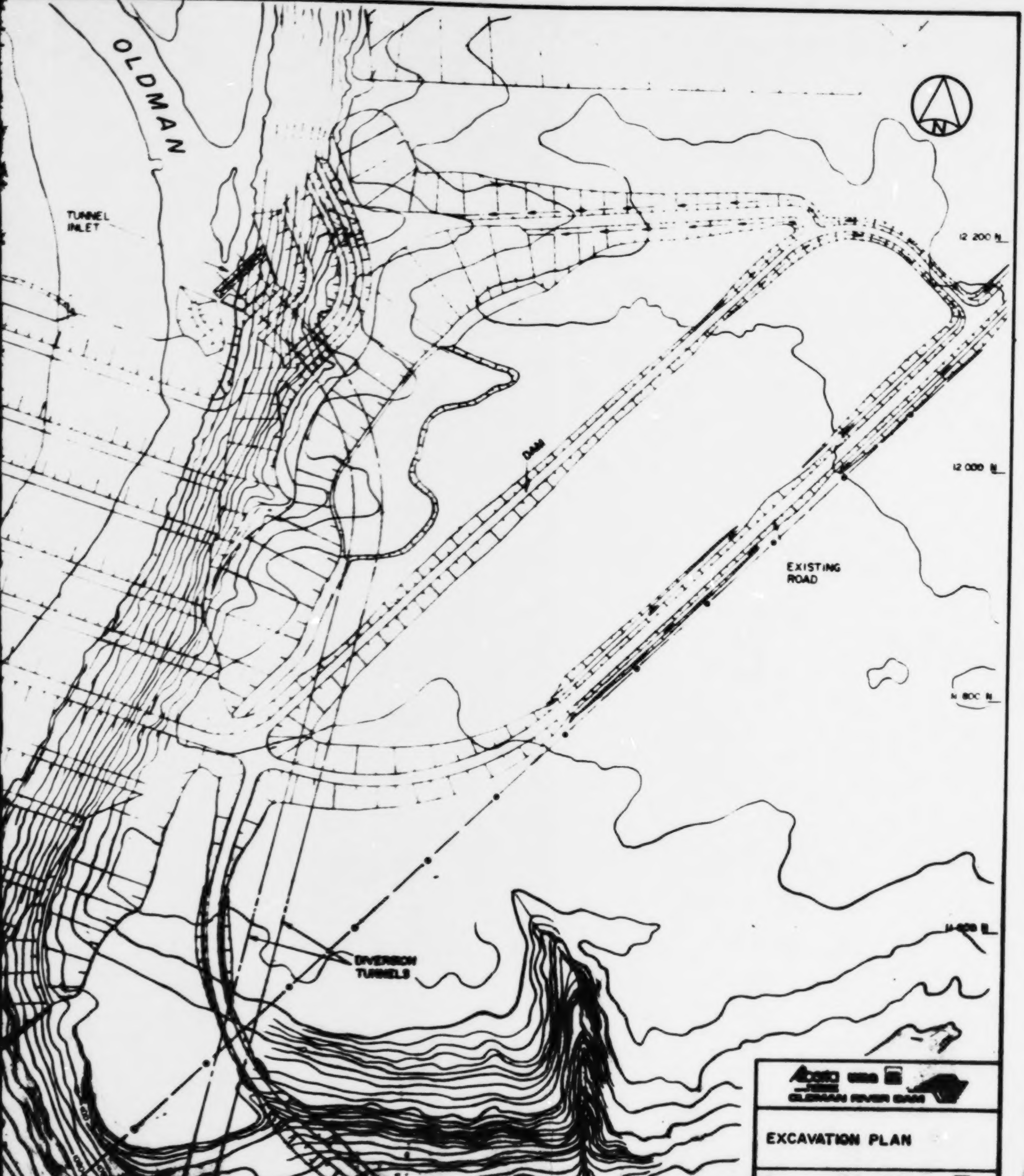
Alberta uma
OLDMAN RIVER DAM

HEADWORKS FOUNDATION
PREDICTED HEAVE/SETTLEMENT

PLATE 6



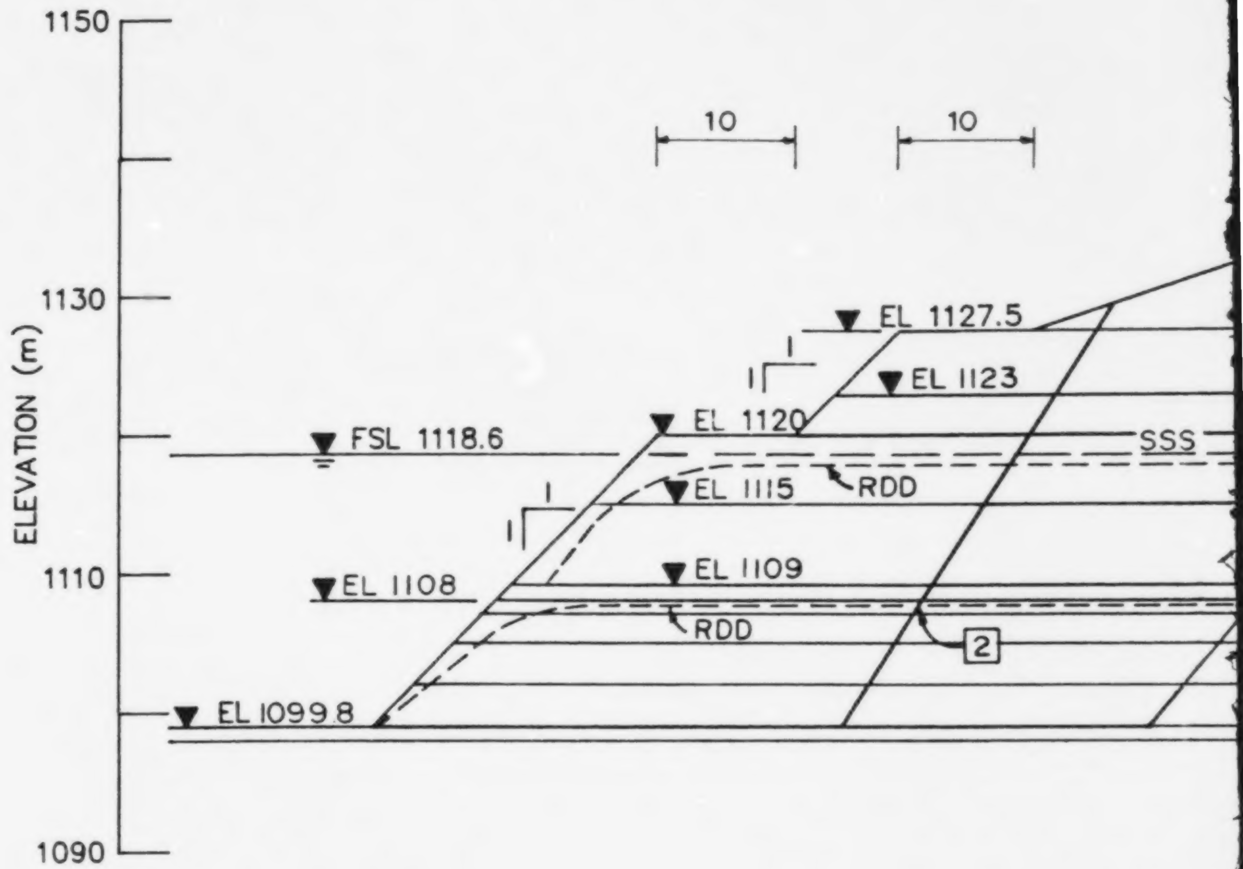
SPILLWAY
APPROACH
CHANNEL
A
SPILLWAY
HEAD WORKS
DAM
TUN
INL
B
C
D
SPILLWAY
CHUTE
RIVER
CONTROL
BUILDING
POOL


OLDMAN
TUNNEL INLET
DAM
EXISTING ROAD
DIVERSION TUNNELS
12 200 N
12 000 N
11 800 N
11 600 N
N

OLDMAN RIVER DAM

EXCAVATION PLAN

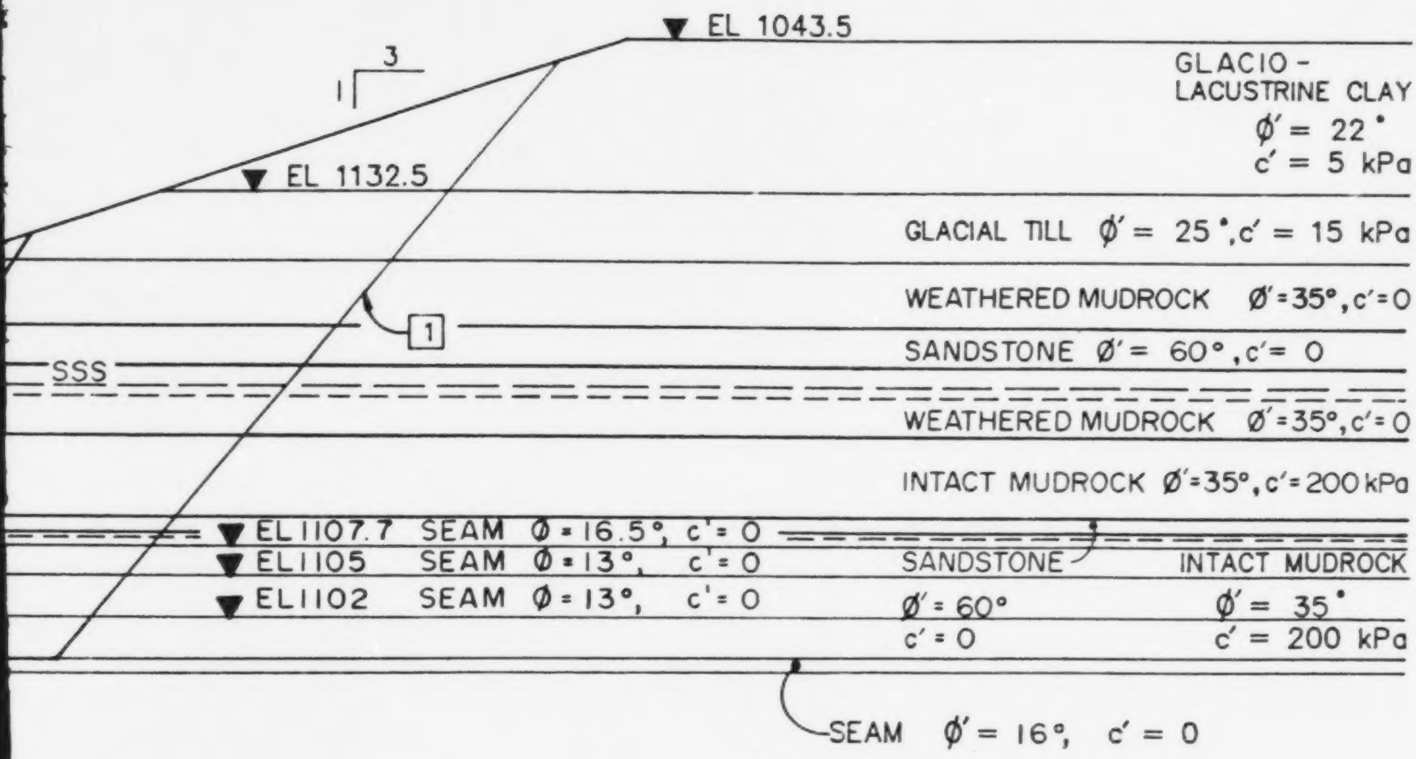
PLATE 7


1150
1130
1110
1090
ELEVATION (m)
10
10
EL 1127.5
1
1
EL 1123
EL 1120
FSL 1118.6
SSS
RDD
EL 1115
1
1
EL 1109
EL 1108
RDD
2
EL 1099.8

SEC

SCALE

FACTOR OF SAFETY F.S. / CASE	MIN. F.S. REQUIRED	SEAM AT EL 1099.8	
		F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS	1.5	1.8	1
2. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL	1.3	1.6	2
3. RAPID RESERVOIR DRAWDOWN FROM FULL SUPPLY LEVEL TO MINIMUM SUPPLY LEVEL PLUS PSEUDO-STATIC SEISMIC LOADING 0.065 g	>1.0	1.4	2


EL 1043.5
3
1
EL 1132.5
GLACIO - LACUSTRINE CLAY
Ø' = 22°
c' = 5 kPa
GLACIAL TILL Ø' = 25°, c' = 15 kPa
WEATHERED MUDROCK Ø' = 35°, c' = 0
1
SANDSTONE Ø' = 60°, c' = 0
SSS
WEATHERED MUDROCK Ø' = 35°, c' = 0
INTACT MUDROCK Ø' = 35°, c' = 200 kPa
EL 1107.7 SEAM Ø = 16.5°, c' = 0
EL 1105 SEAM Ø = 13°, c' = 0
EL 1102 SEAM Ø = 13°, c' = 0
SANDSTONE
Ø' = 60°
c' = 0
INTACT MUDROCK
Ø' = 35°
c' = 200 kPa
SEAM Ø' = 16°, c' = 0

SECTION A

SCALE 1:500

LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

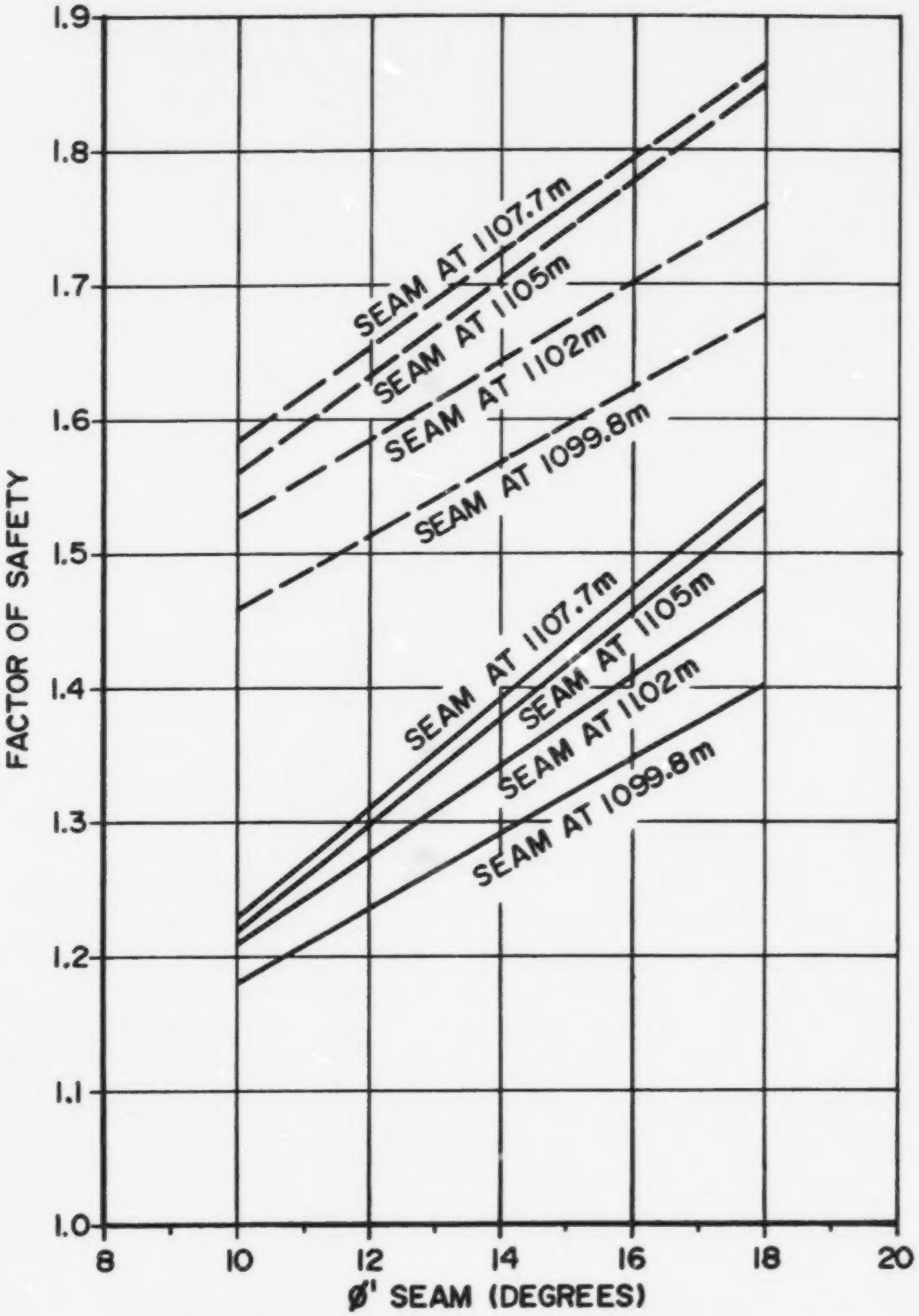
RDD – ASSUMED PHREATIC SURFACE DURING RAPID DRAWDOWN

SSS – PHREATIC SURFACE DURING STEADY STATE CONDITIONS

OLDMAN RIVER DAM

APPROACH CHANNEL
SIDE SLOPES
STABILITY – I

PLATE 8



SECTION A
APPROACH CHANNEL
SIDE SLOPE STABILITY
RAPID DRAWDOWN

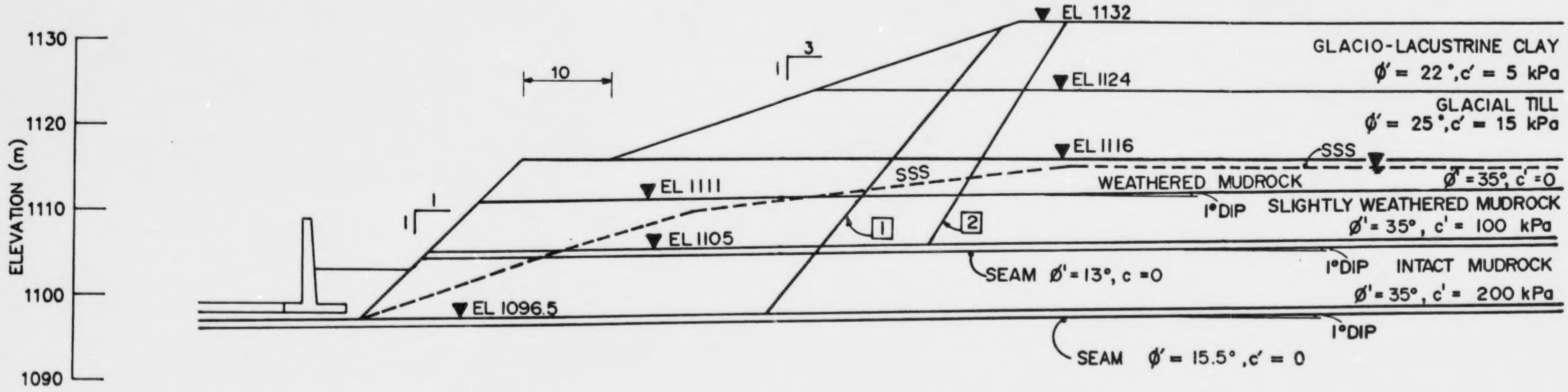
LEGEND

— — — —WITHOUT EARTHQUAKE
————WITH EARTHQUAKE (0.065 g)

Alberta
OLDMAN RIVER DAM

APPROACH CHANNEL
SIDE SLOPES
STABILITY – 2

PLATE 9


EL 1132
GLACIO-LACUSTRINE CLAY
Ø' = 22°, c' = 5 kPa
EL 1124
GLACIAL TILL
Ø' = 25°, c' = 15 kPa
EL 1116
SSS
Ø' = 35°, c' = 0
WEATHERED MUDROCK
EL 1111
SSS
1°DIP
SLIGHTLY WEATHERED MUDROCK
Ø' = 35°, c' = 100 kPa
1
2
EL 1105
SEAM Ø' = 13°, c = 0
1°DIP
INTACT MUDROCK
Ø' = 35°, c' = 200 kPa
EL 1096.5
1°DIP
SEAM Ø' = 15.5°, c' = 0
10
1
3
1
1
1130
1120
1110
1100
1090
ELEVATION (m)

SECTION B

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1096.5m		SEAM AT EL. 1105m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE	1.5	1.5	1	1.4	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE SEEPAGE CONDITIONS AND PSEUDO-STATIC SEISMIC LOADING 0.065g	> 1.0	1.1	1	1.1	2

LEGEND:

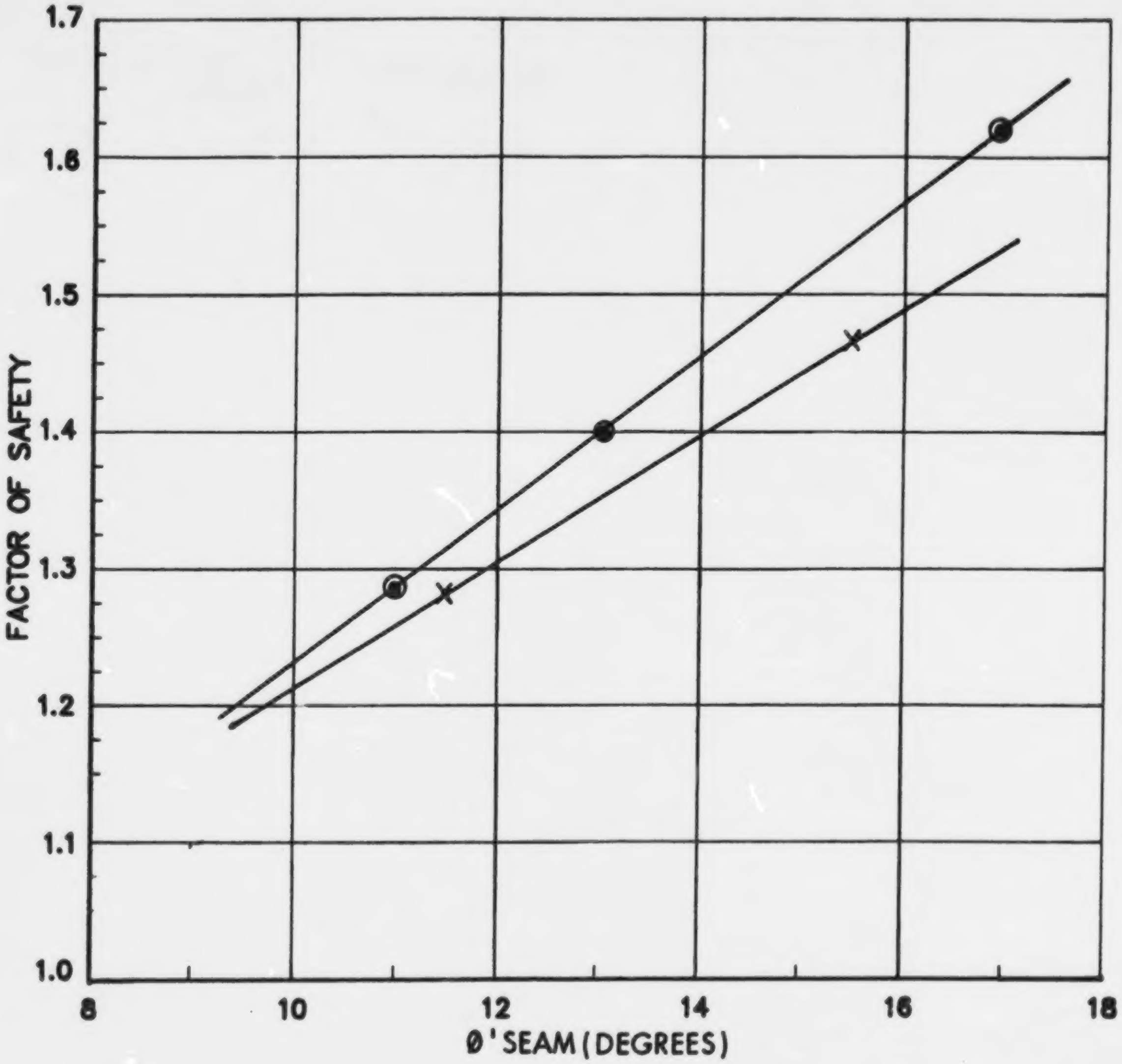
SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

Alberta uma

OLDMAN RIVER DAM

CHUTE
SIDE SLOPES
STABILITY – 1

PLATE 10



SECTION B

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

× – 1096.5 m

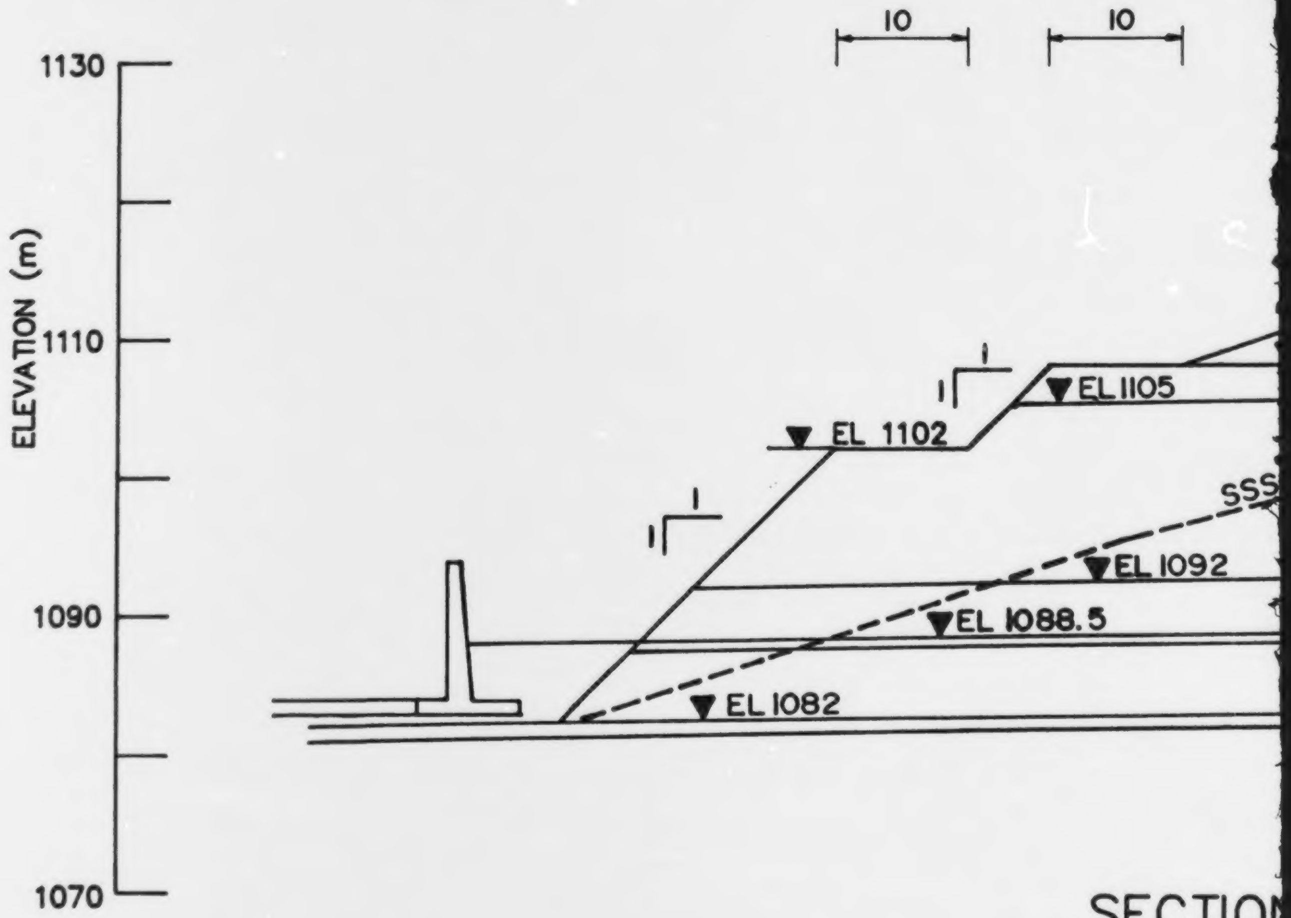
● – 1105 m

Alberta

OLDMAN RIVER DAM

CHUTE
SIDE SLOPES
STABILITY – 2

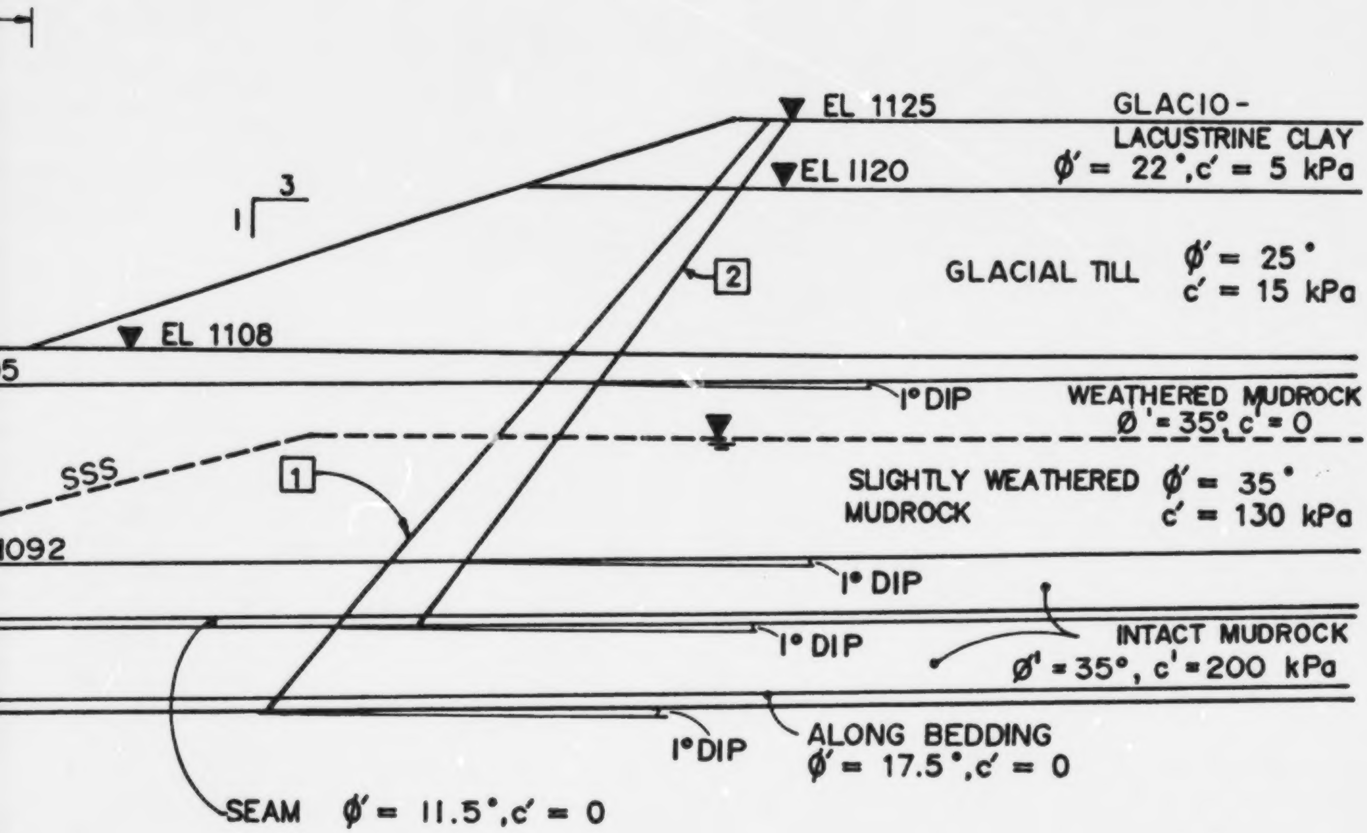
PLATE 11


10
10
1130
1110
1090
1070
ELEVATION (m)
1
1
1
1
EL 1105
EL 1102
SSS
EL 1092
EL 1088.5
EL 1082

SECTIO

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL. 1082 m		SEAM AT EL 1088.5m	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS	1.5	1.6	1	1.5	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS PLUS PSEUDO-STATIC SEISMIC LOADING 0.065g	> 1.0	1.3	1	1.1	2



EL 1125
GLACIO-LACUSTRINE CLAY
$\phi' = 22°, c' = 5$ kPa
EL 1120
1
3
2
GLACIAL TILL
$\phi' = 25°$
$c' = 15$ kPa
EL 1108
5
1° DIP
WEATHERED MUDROCK
$\phi' = 35°, c' = 0$
SSS
1
SLIGHTLY WEATHERED MUDROCK
$\phi' = 35°$
$c' = 130$ kPa
1092
1° DIP
1° DIP
INTACT MUDROCK
$\phi' = 35°, c' = 200$ kPa
1° DIP
ALONG BEDDING
$\phi' = 17.5°, c' = 0$
SEAM $\phi' = 11.5°, c' = 0$

CTION C

1:500

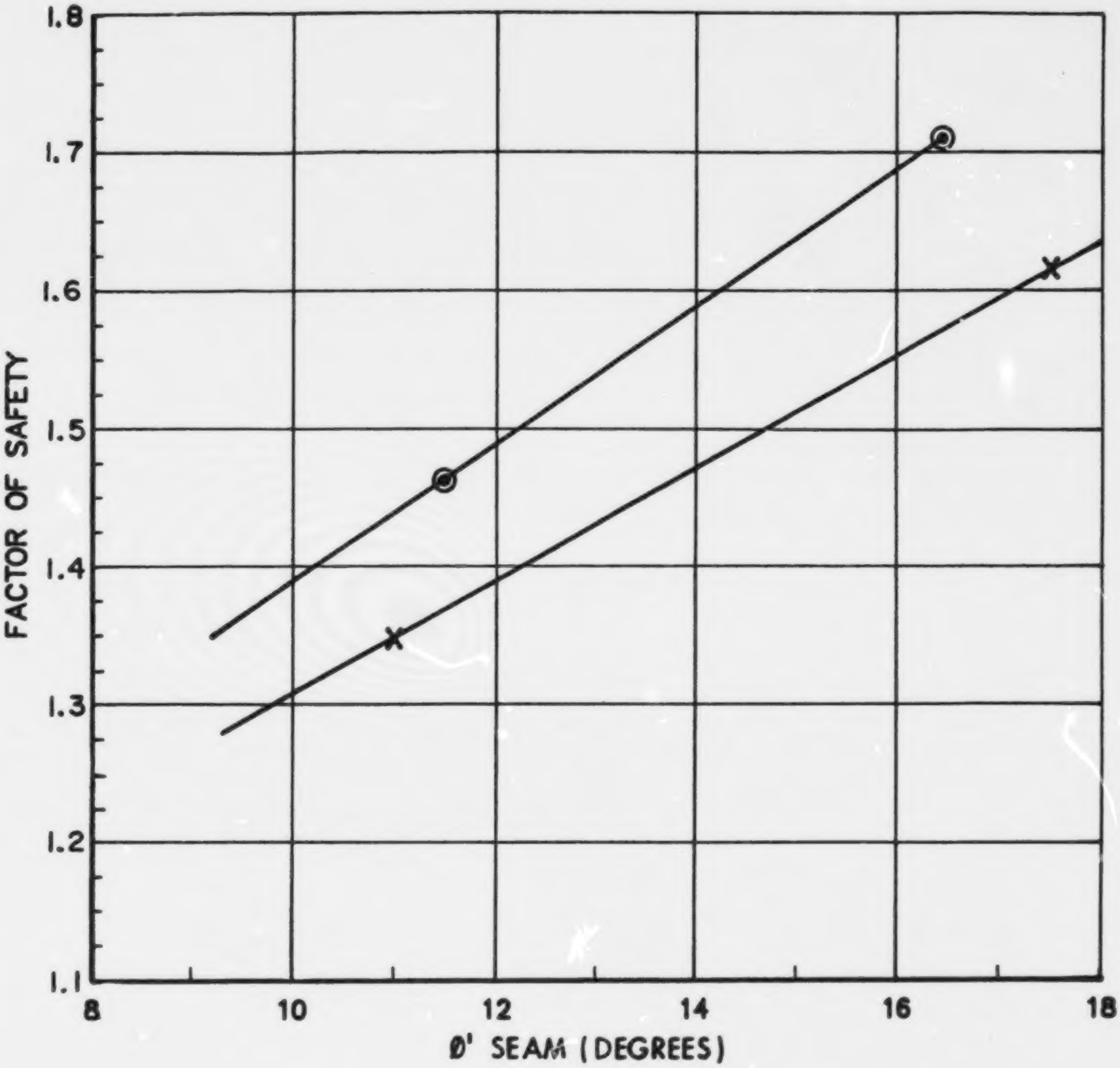
LEGEND:

SSS – ASSUMED PHREATIC SURFACE
DURING STEADY STATE SEEPAGE

OLDMAN RIVER DAM

CHUTE
SIDE SLOPES
STABILITY - 3

PLATE 12



SECTION C

FACTOR OF SAFETY VS ϕ' SEAM

FAILURE THROUGH SEAM AT ELEVATION:

× – 1082 m

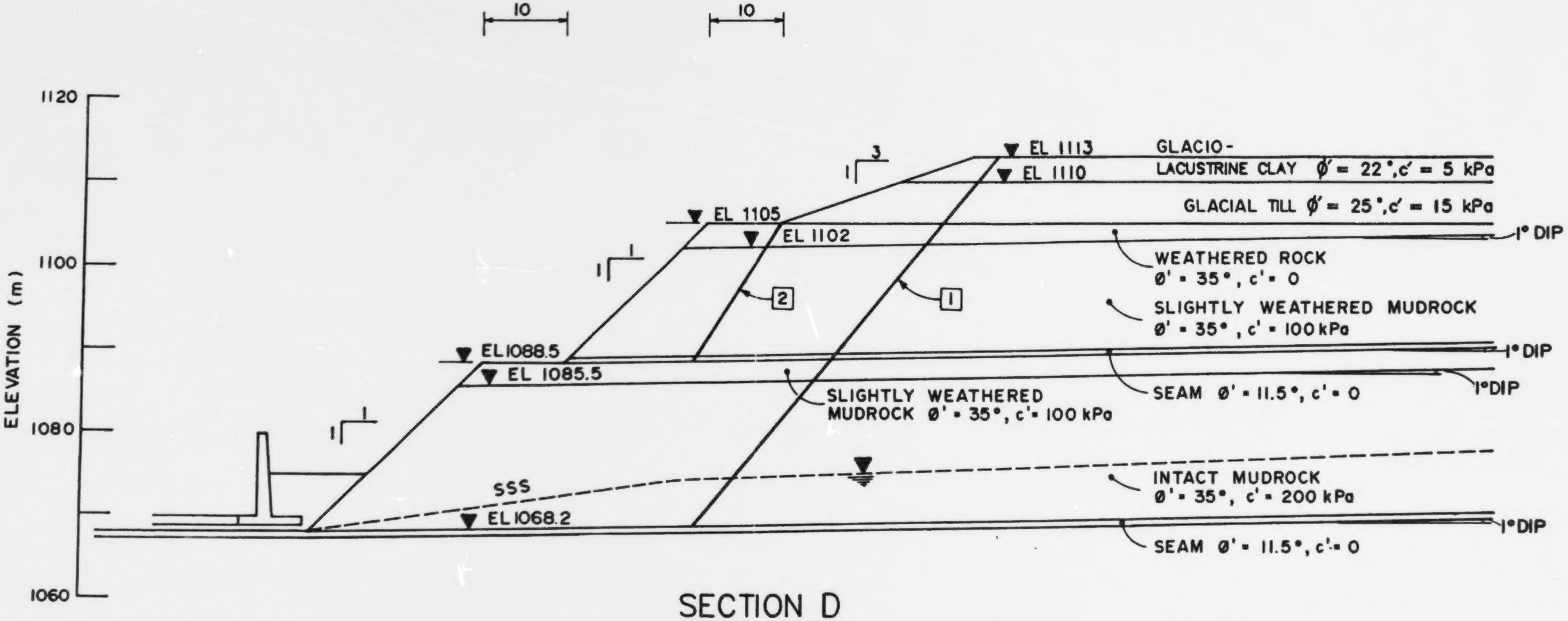
◎ – 1088.5 m

Alberta

OLDMAN RIVER DAM

CHUTE
SIDE SLOPES
STABILITY – 4

PLATE 13


10
10
1120
1100
1080
1060
ELEVATION (m)
1
3
EL 1113
EL 1110
GLACIO-
LACUSTRINE CLAY Ø' = 22°, c' = 5 kPa
GLACIAL TILL Ø' = 25°, c' = 15 kPa
EL 1105
EL 1102
1° DIP
1
1
WEATHERED ROCK
Ø' = 35°, c' = 0
2
1
SLIGHTLY WEATHERED MUDROCK
Ø' = 35°, c' = 100 kPa
EL 1088.5
1° DIP
EL 1085.5
SEAM Ø' = 11.5°, c' = 0
1° DIP
1
1
SLIGHTLY WEATHERED
MUDROCK Ø' = 35°, c' = 100 kPa
SSS
INTACT MUDROCK
Ø' = 35°, c' = 200 kPa
EL 1068.2
1° DIP
SEAM Ø' = 11.5°, c' = 0

SECTION D

SCALE 1:500

CASE \ FACTOR OF SAFETY F.S.	MIN. F.S. REQUIRED	SEAM AT EL.1068.2		SEAM AT EL.1088.5	
		F.S.	FAILURE SURFACE	F.S.	FAILURE SURFACE
1. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS	1.5	1.5	1	1.7	2
2. RESERVOIR AT FULL SUPPLY LEVEL WITH STEADY STATE CONDITIONS PLUS PSEUDO-STATIC SEISMIC LOADING 0.065g	>1.0	1.2	1	1.4	2

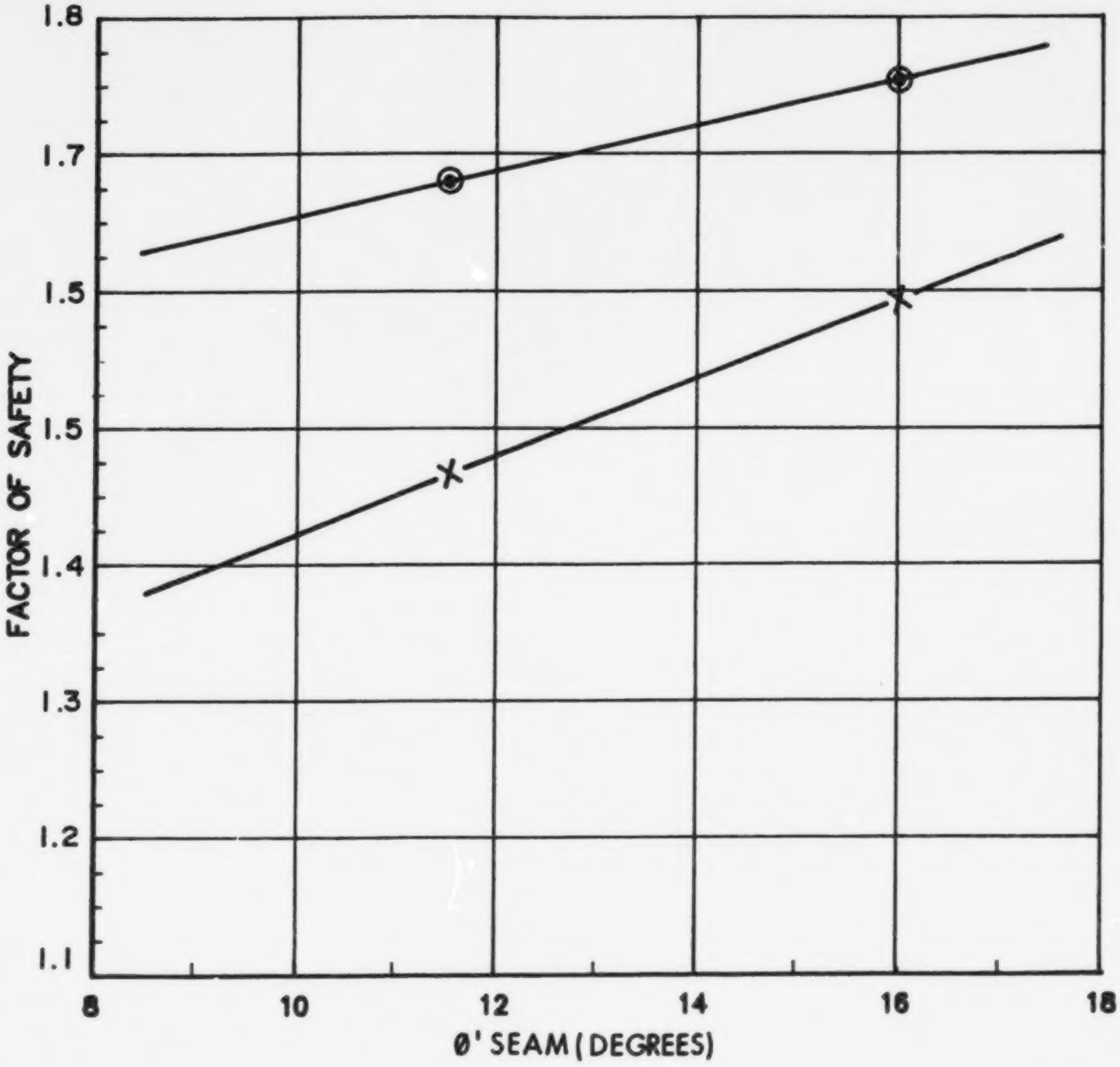
LEGEND:

SSS – ASSUMED PHREATIC SURFACE DURING STEADY STATE SEEPAGE

Alberta
OLDMAN RIVER DAM

CHUTE
SIDE SLOPES
STABILITY – 5

PLATE 14



SECTION D

FACTOR OF SAFETY VS Ø' SEAM

FAILURE THROUGH
SEAM AT ELEVATION:

● - 1088.5m

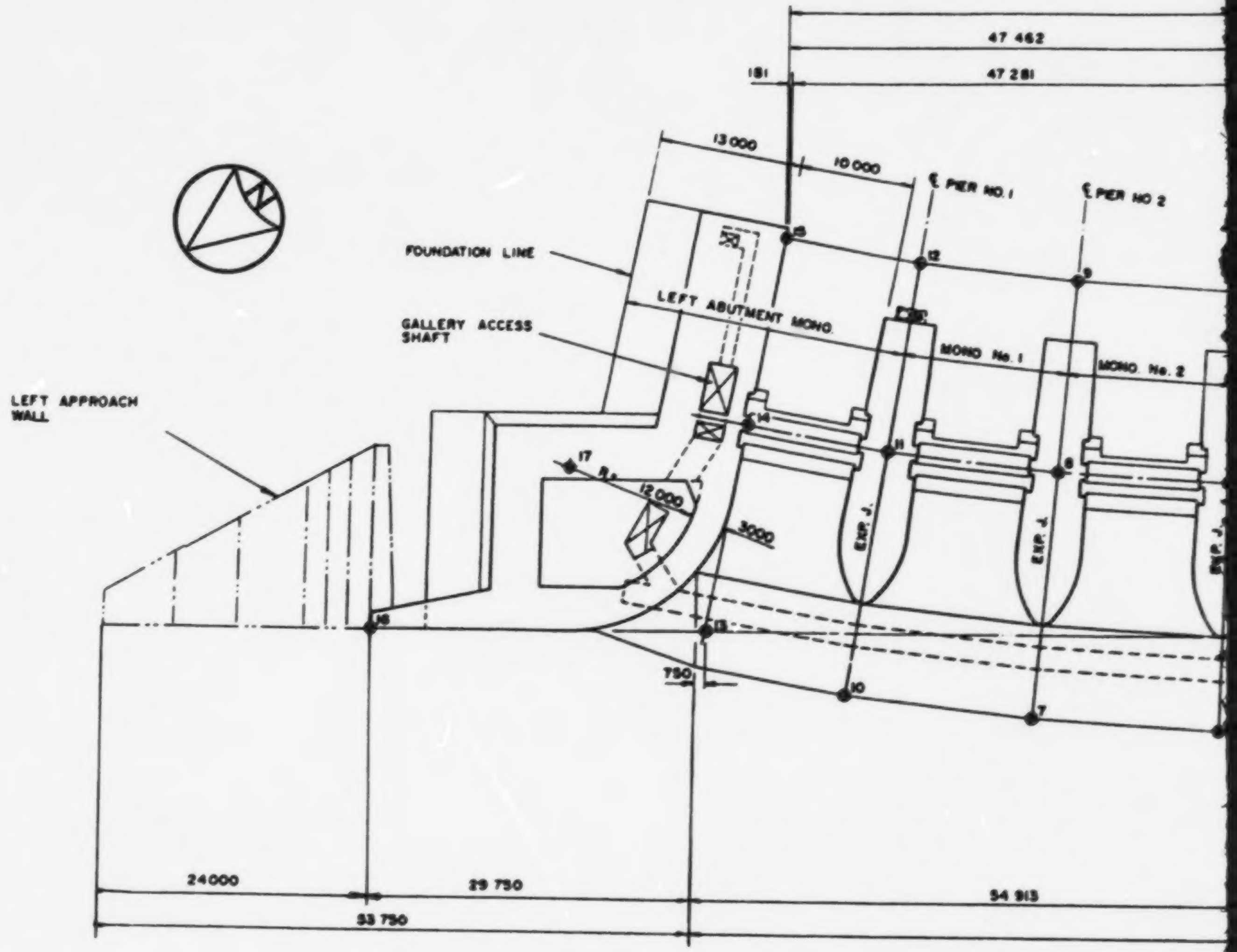
× - 1068.2m

Alberta

OLDMAN RIVER DAM

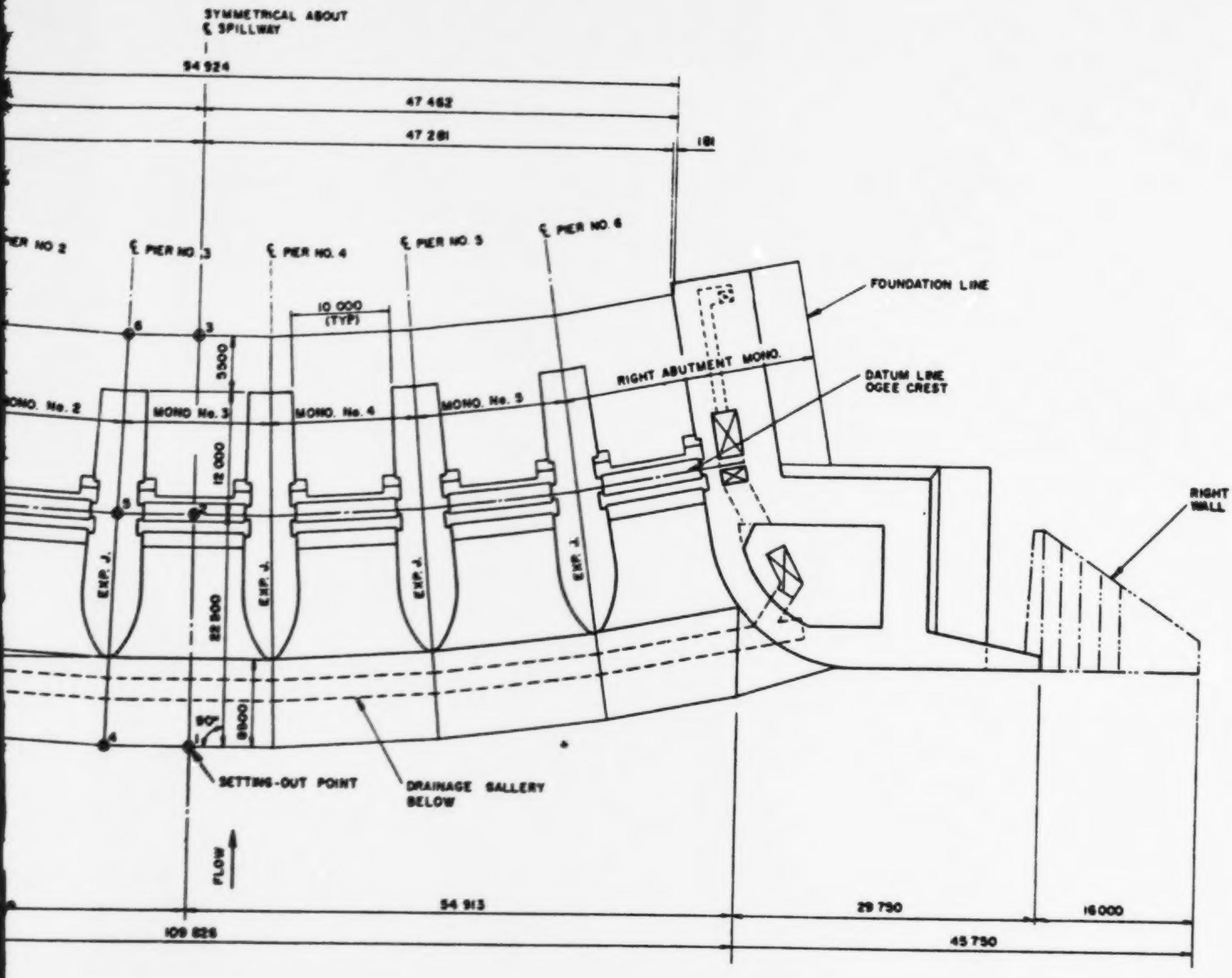
CHUTE
SIDE SLOPES
STABILITY - 6

PLATE 15


47 462
47 281
181
13 000
10 000
℄ PIER NO. 1
℄ PIER NO 2
FOUNDATION LINE
LEFT ABUTMENT MONO.
MONO. No. 1
MONO. No. 2
GALLERY ACCESS SHAFT
LEFT APPROACH WALL
15
12
9
14
11
8
17
R
12 000
3000
EXP. J.
EXP. J.
EXP. J.
16
13
750
10
7
24000
29 750
53 750
54 913

Pl

5


SYMMETRICAL ABOUT
℄ SPILLWAY
54 924
47 462
47 281
181
PIER NO 2
℄ PIER NO. 3
℄ PIER NO. 4
℄ PIER NO. 5
℄ PIER NO. 6
FOUNDATION LINE
10 000
(TYP)
3500
RIGHT ABUTMENT MONO.
DATUM LINE
OGEE CREST
MONO. No. 2
MONO No. 3
MONO. No. 4
MONO. No. 5
12 000
RIGHT APPROACH
WALL
EXP. J.
EXP. J.
EXP. J.
EXP. J.
22 500
90°
9500
SETTING-OUT POINT
DRAINAGE GALLERY
BELOW
FLOW
54 913
29 750
16 000
109 826
45 750

PLAN AT EL 1112.000

0
5
10
15m
5
2.5

OLDMAN RIVER DAM

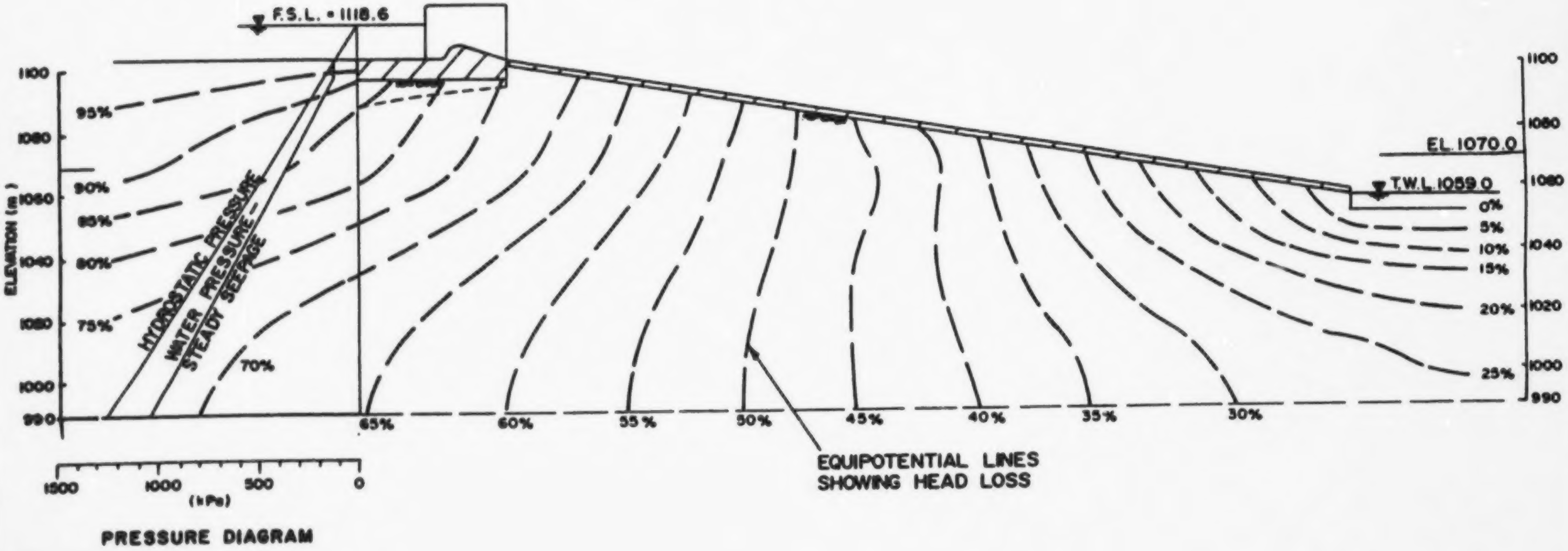
HEADWORKS
GENERAL ARRANGEMENT

PLATE 16

NOTE:
Kh = 10 Kv
NO DRAINAGE TUNNEL
NO GROUT CURTAIN

MATERIAL PERMEABILITIES (m/sec.)

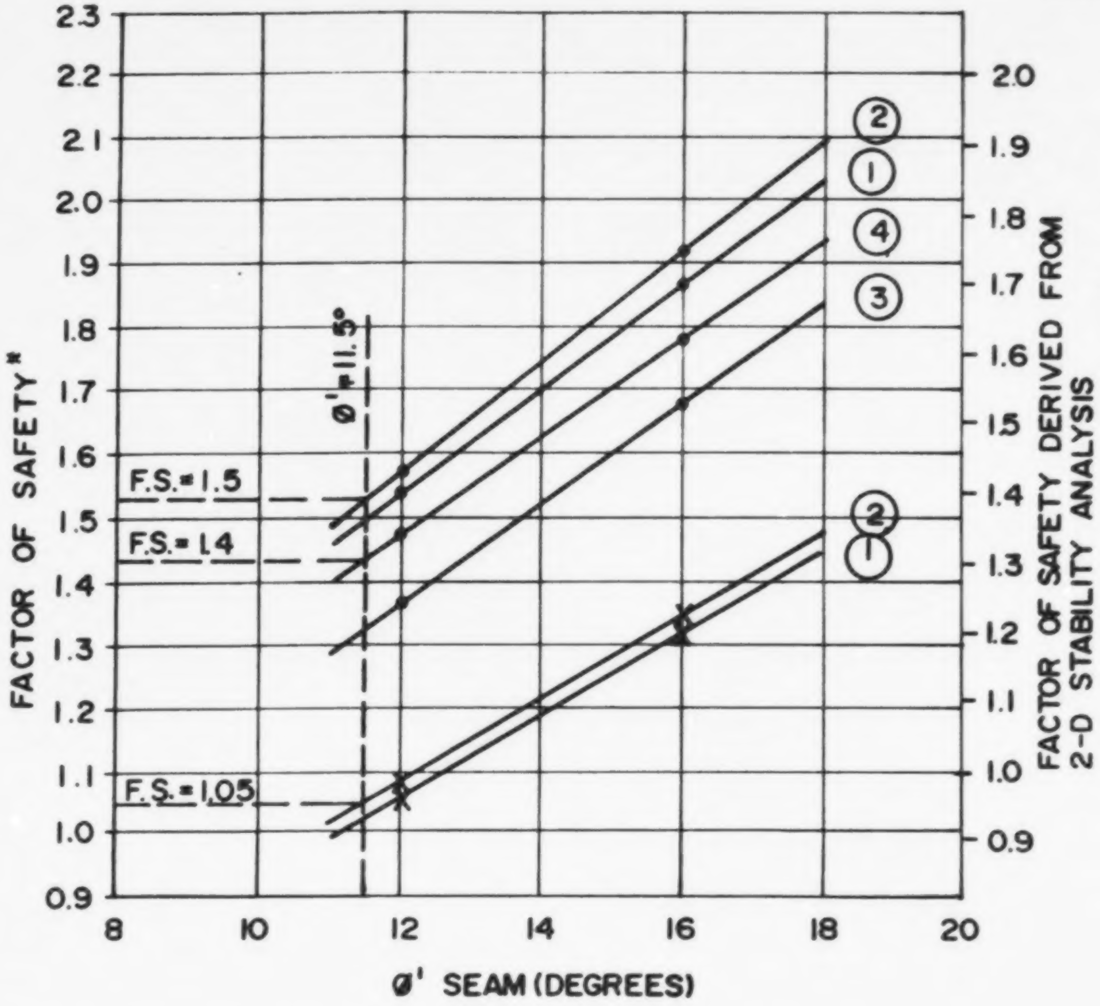
	Kh	Kv
ABOVE ELEV. 1070	1×10^{-6}	1×10^{-7}
BELOW ELEV. 1070	1×10^{-7}	1×10^{-8}


F.S.L. = 1118.6
EL. 1070.0
T.W.L. 1059.0
ELEVATION (m)
1100
1080
1060
1040
1020
1000
990
95%
90%
85%
80%
75%
70%
65%
60%
55%
50%
45%
40%
35%
30%
25%
20%
15%
10%
5%
0%
HYDROSTATIC PRESSURE
WATER PRESSURE - STEADY SEEPAGE
EQUIPOTENTIAL LINES SHOWING HEAD LOSS
1500
1000
500
0
(kPa)
PRESSURE DIAGRAM

Alberta
OLDMAN RIVER DAM

HEADWORKS
SEEPAGE ANALYSIS

PLATE 17

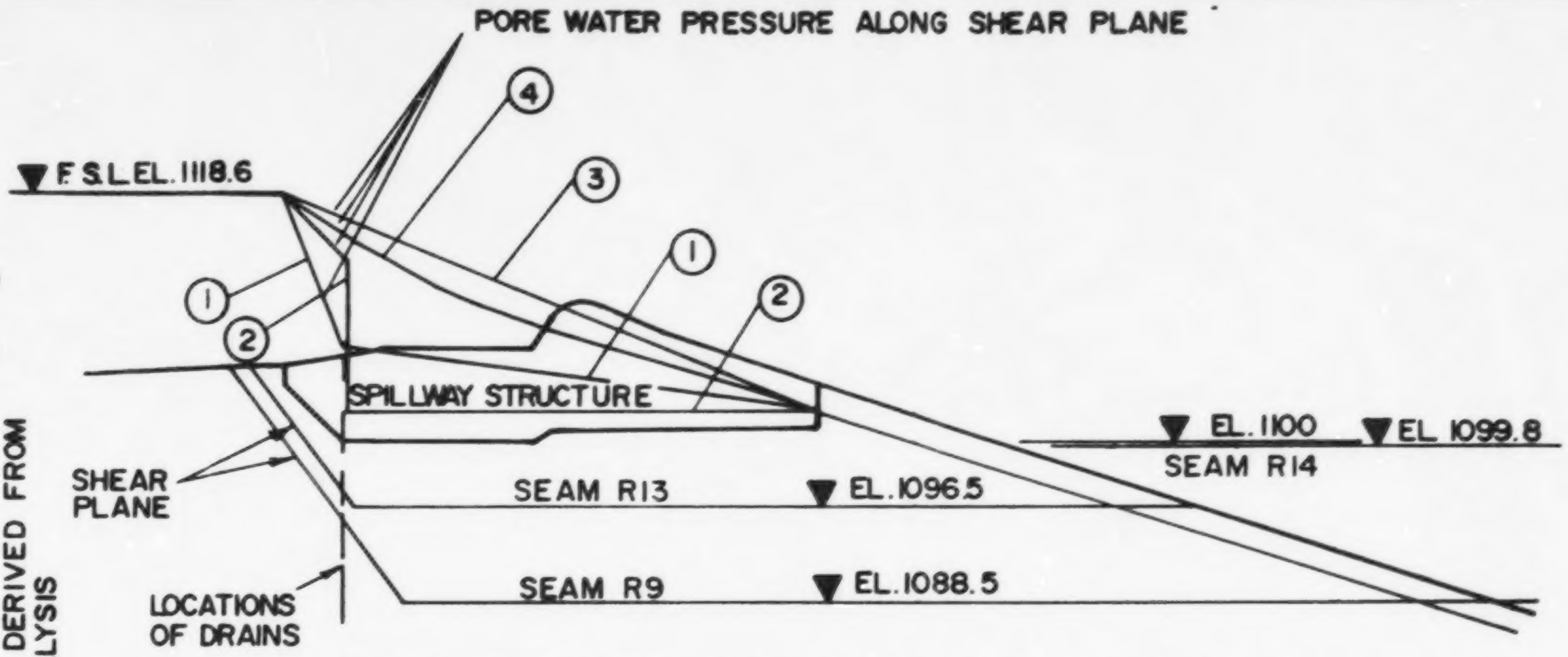


FACTOR OF SAFETY vs Ø' SEAM FOR DIFFERENT PORE WATER PRESSURES SLIDING THROUGH SEAM R9 AT ELEVATION 1088.5 m.

*NOTE· THE FACTOR OF SAFETY IS BASED ON THE RESULTS OF 2-D STABILITY ANALYSIS AND INCLUDES 3-D EFFECT.

LEGEND

—●—●— RESERVOIR AT FSL, GATE CLOSED
—✕—✕— RESERVOIR AT FSL, GATE CLOSED HORIZONTAL SEISMIC LOADING OF 0.065g

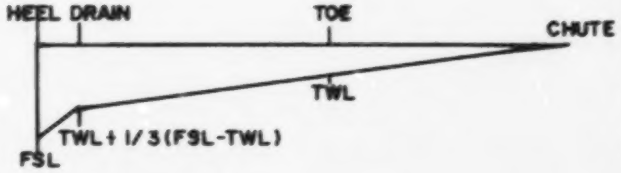

PORE WATER PRESSURE ALONG SHEAR PLANE
④
③
①
②
①
②
F.S.L. EL. 1118.6
SPILLWAY STRUCTURE
SHEAR PLANE
LOCATIONS OF DRAINS
EL. 1100
EL. 1099.8
SEAM R14
SEAM R13
EL. 1096.5
SEAM R9
EL. 1088.5

PORE WATER PRESSURES ALONG SHEAR PLANE

NOTE· 1. CROSS BEDDING SHEAR STRENGTH OF ROCK BETWEEN GROUND SURFACE AND SEAM IS C' = 50 kPa Ø' = 35°.

2. FOR SLIDING THROUGH SEAMS R13 AND R14 ELEVATIONS 1096.5 m AND 1099.8 m SEE PLATES 19 AND 20.

LEGEND·


HEEL DRAIN
TOE
CHUTE
TWL
TWL + 1/3 (FSL-TWL)
FSL

① HEAD DROPS FROM FSL AT HEEL TO [TWL + 1/3 (FSL - TWL)] AT DRAINS AND TO TWL AT TOE.

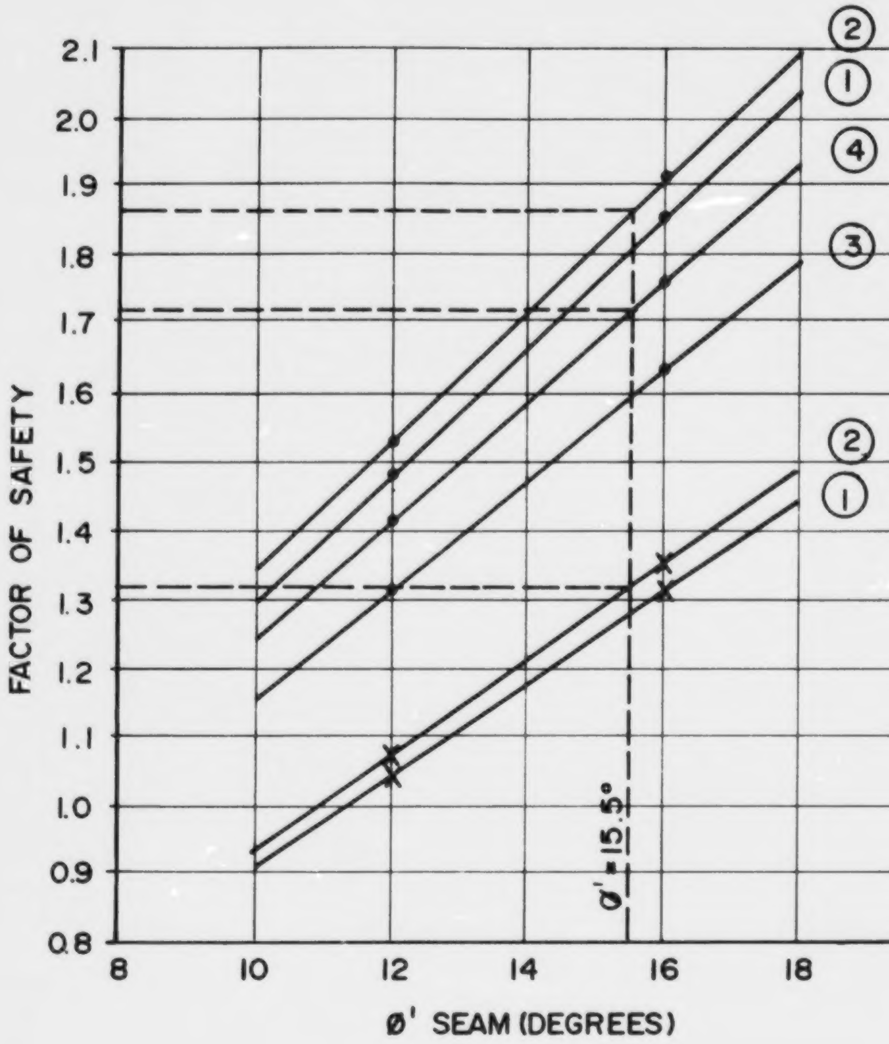
② FINITE ELEMENT SEEPAGE ANALYSIS, GROUT CURTAIN AND DRAINS EFFECTIVE BETWEEN THE DRAINS AND TOE, THE PHREATIC SURFACE IS AT THE TAILWATER LEVEL.

③ LINEAR VARIATION FROM FSL AT HEEL TO TWL AT TOE.

④ FINITE ELEMENT SEEPAGE ANALYSIS, INEFFECTIVE GROUT CURTAIN, DRAINS BLOCKED.

HEADWORKS
STABILITY – EL. 1088.5m

PLATE 18



FACTOR OF SAFETY vs Ø' SEAM FOR DIFFERENT PORE WATER PRESSURES SLIDING THROUGH SEAM R13 AT ELEVATION 1096.5

NOTE: FOR DESCRIPTION OF LOADING CONDITIONS SEE PLATE 18

LEGEND

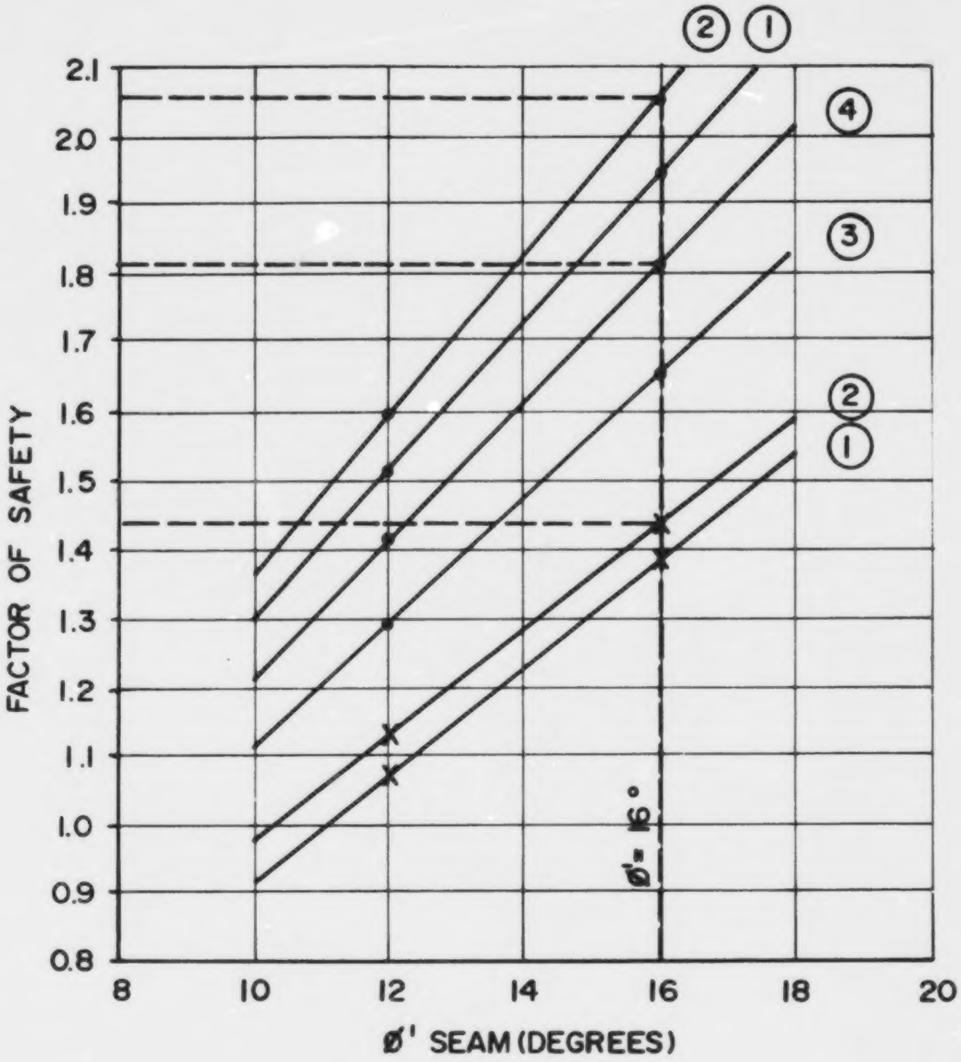
—●—●— RESERVOIR AT FSL, GATES CLOSED

—✕—✕— RESERVOIR AT FSL, GATES CLOSED, HORIZONTAL SEISMIC LOADING OF 0.065g

OLDMAN RIVER DAM

HEADWORKS
STABILITY – EL. 1096.5m

PLATE 19



FACTOR OF SAFETY vs Ø' SEAM FOR DIFFERENT PORE WATER PRESSURES SLIDING THROUGH SEAM R14 AT ELEVATION 1099.8

NOTE: FOR DESCRIPTION OF LOADING CONDITIONS SEE PLATE 18

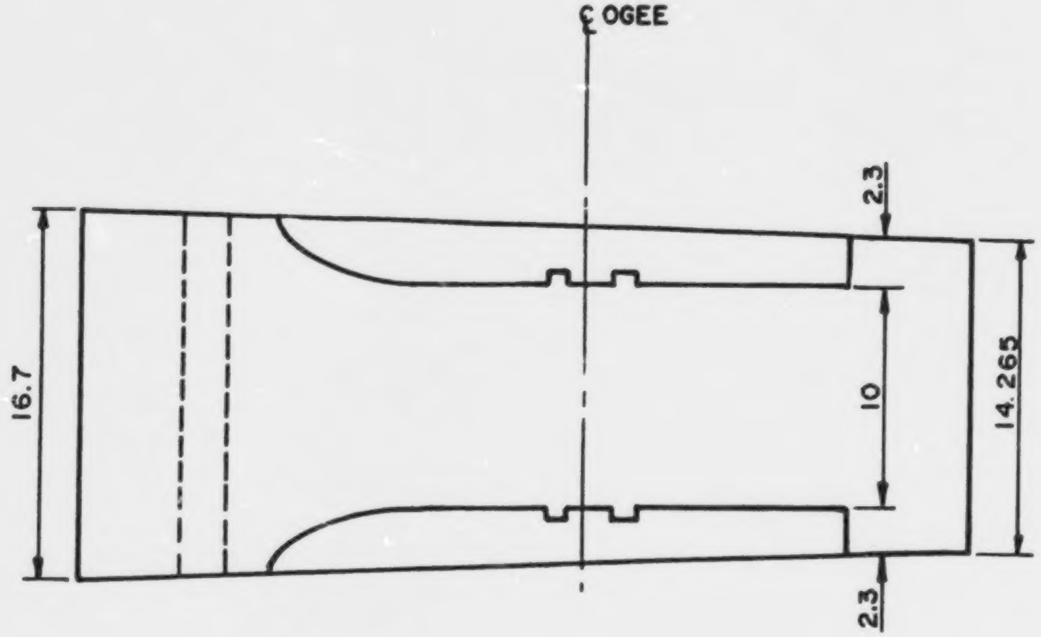
LEGEND

•—• RESERVOIR AT FSL, GATES CLOSED

×—× RESERVOIR AT FSL, GATES CLOSED, HORIZONTAL SEISMIC LOADING OF 0.065g

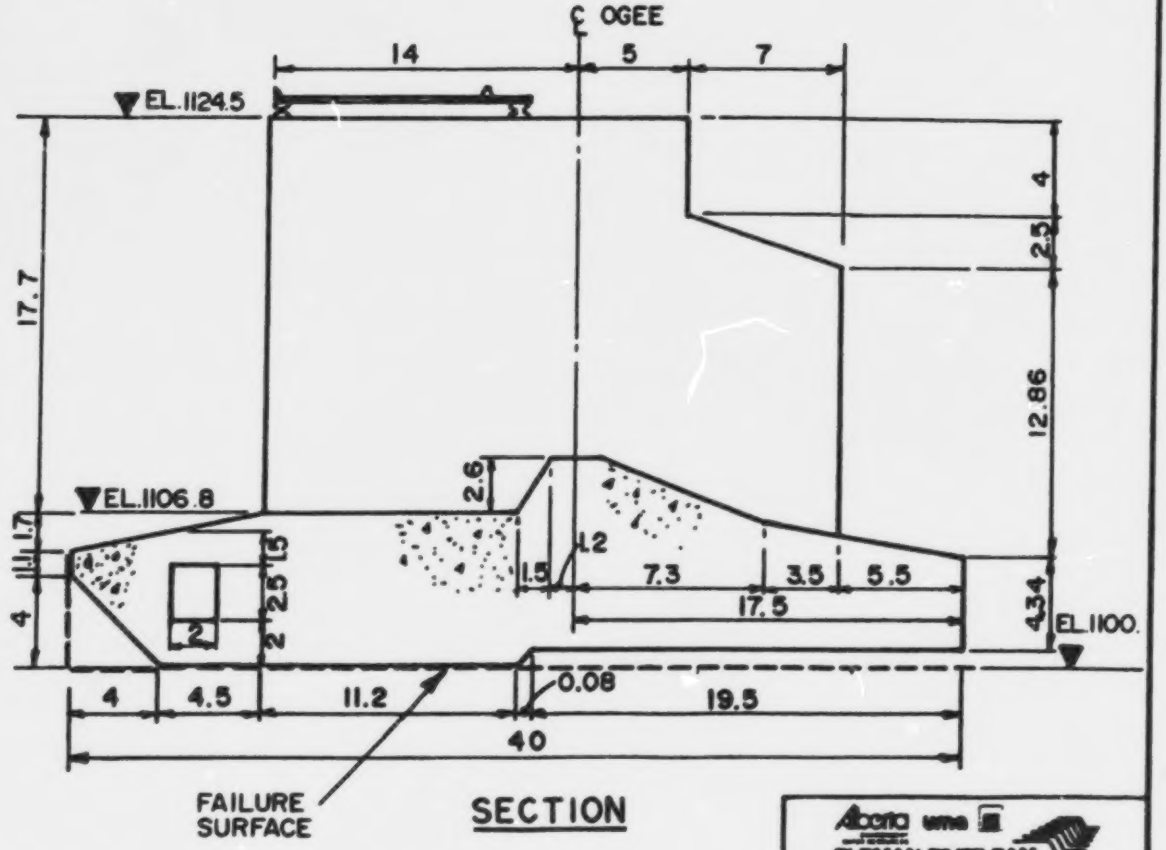
HEADWORKS
STABILITY – EL. 1099.8 m

PLATE 20



℄ OGEE
2.3
16.7
10
14.265
2.3

PLAN



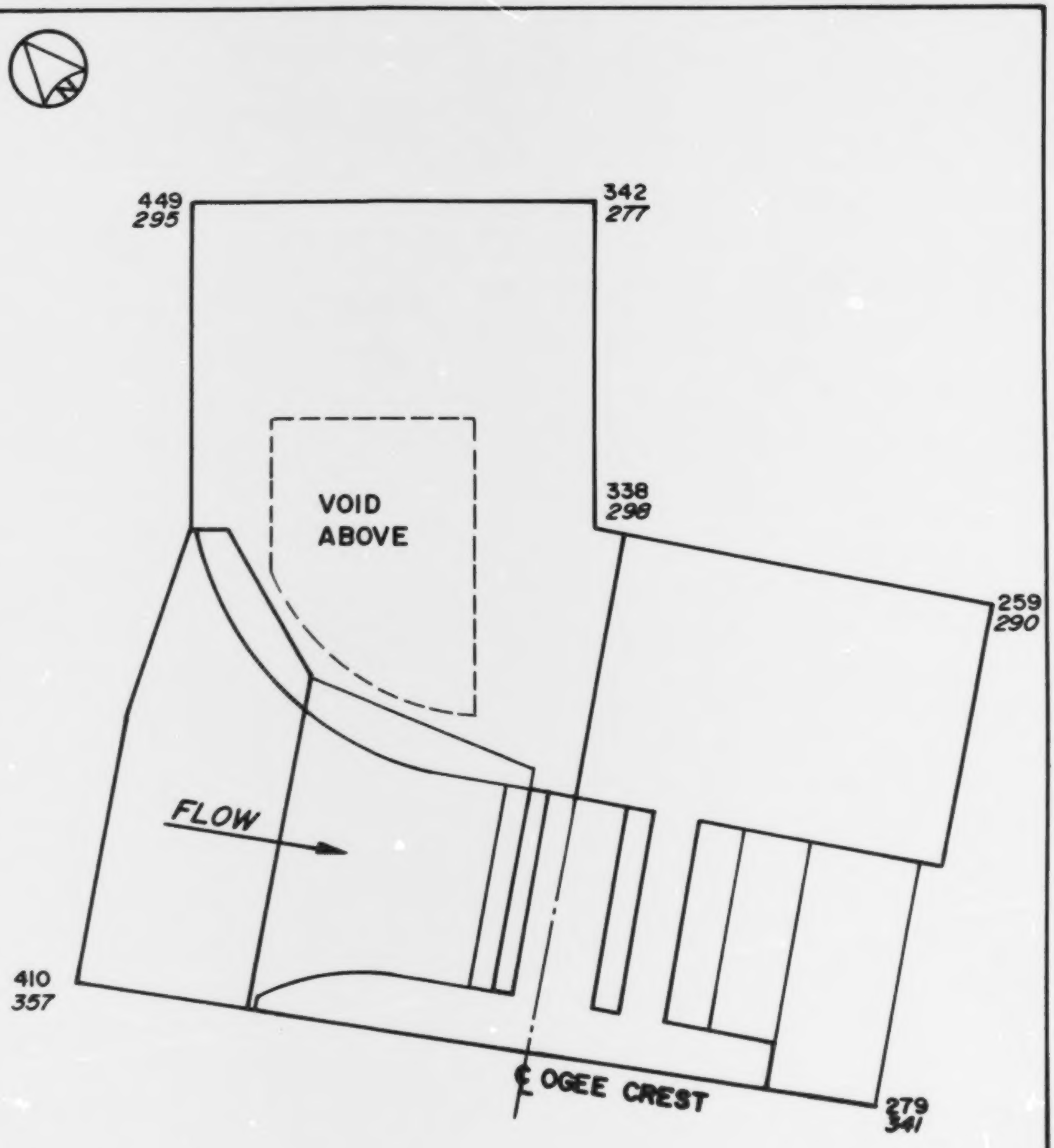
℄ OGEE
14
5
7
EL.1124.5
17.7
4
2.5
12.86
2.6
EL.1106.8
1.7
1.5
2.5
2
2
4
1.5
1.2
7.3
3.5
5.5
17.5
4.34
EL.1100.
4
4.5
11.2
0.08
19.5
40
FAILURE SURFACE

SECTION

Alberta
OLDMAN RIVER DAM

TYPICAL WEIR MONOLITH GEOMETRY

PLATE 21


N
449
295
342
277
VOID
ABOVE
338
298
259
290
FLOW
410
357
℄ OGEE CREST
279
341

ABUTMENT FOUNDATION BEARING PRESSURES (kPa)

0 2 4 6m
2 1

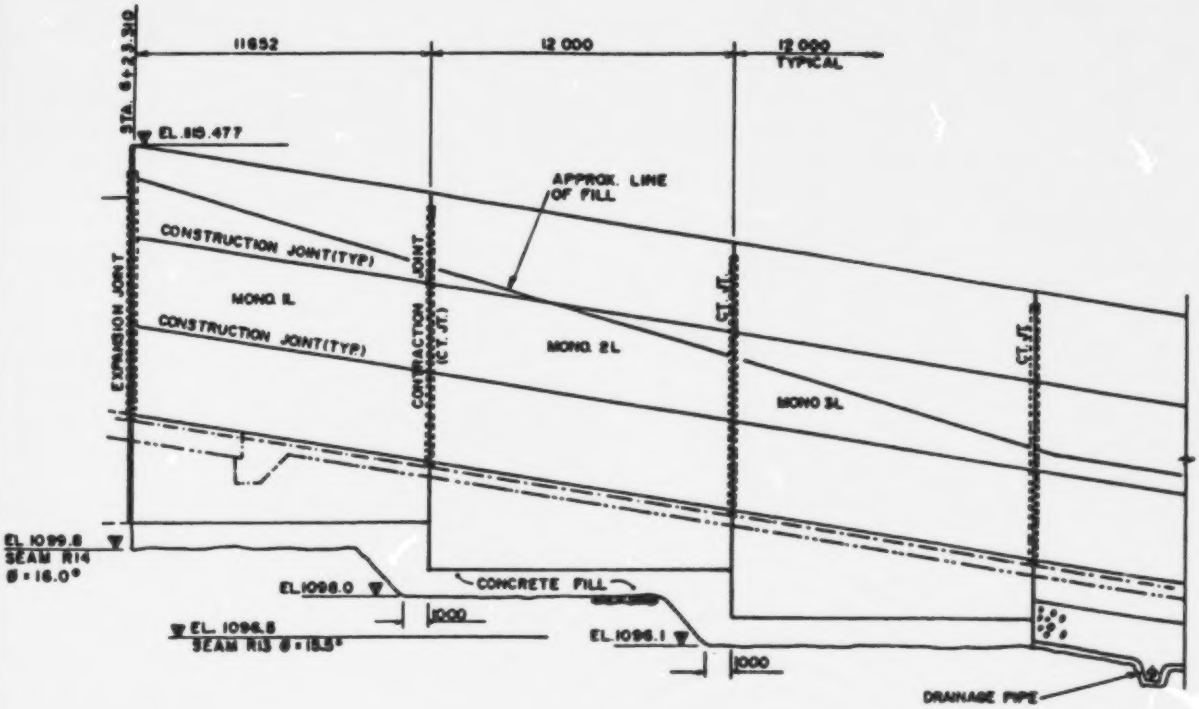
(LEFT ABUTMENT SHOWN, RIGHT ABUTMENT
SIMILAR OPPOSITE HAND)

410 DRY CONDITION
357 FSL

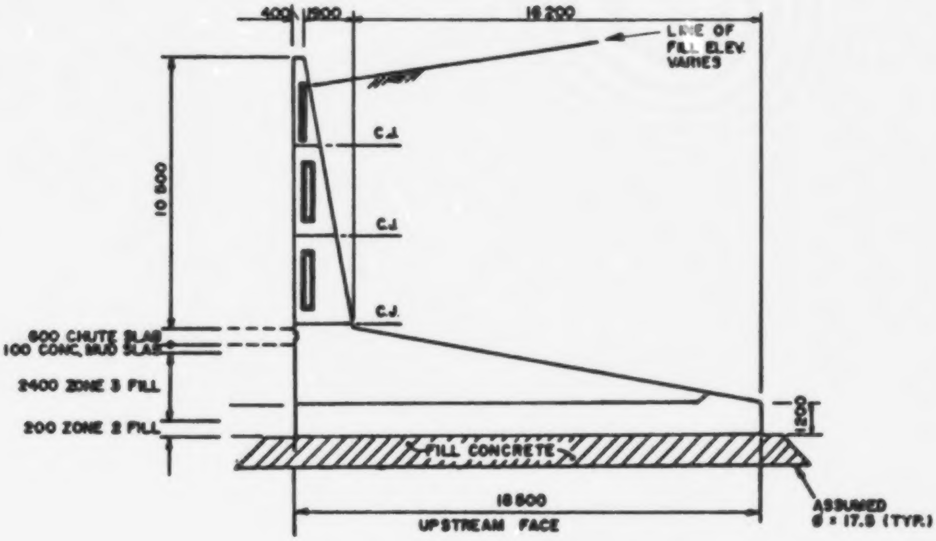
OLDMAN RIVER DAM

**ABUTMENT FOUNDATION
BEARING PRESSURES**

PLATE 22


STA. 6+23.310
11652
12 000
12 000 TYPICAL
EL. 815.477
APPROX. LINE OF FILL
CONSTRUCTION JOINT(TYP)
MONO. 1L
CONSTRUCTION JOINT(TYP.)
EXPANSION JOINT
CONTRACTION JOINT (CT. JT.)
MONO. 2L
C.T. JT.
MONO 3L
C.T. JT.
EL. 1099.8 SEAM R14 Ø = 16.0°
EL.1098.0
CONCRETE FILL
1000
EL. 1096.8 SEAM R13 Ø = 15.5°
EL.1096.1
1000
DRAINAGE PIPE

ELEVATION - MONOLITHS 1, 2 & 3

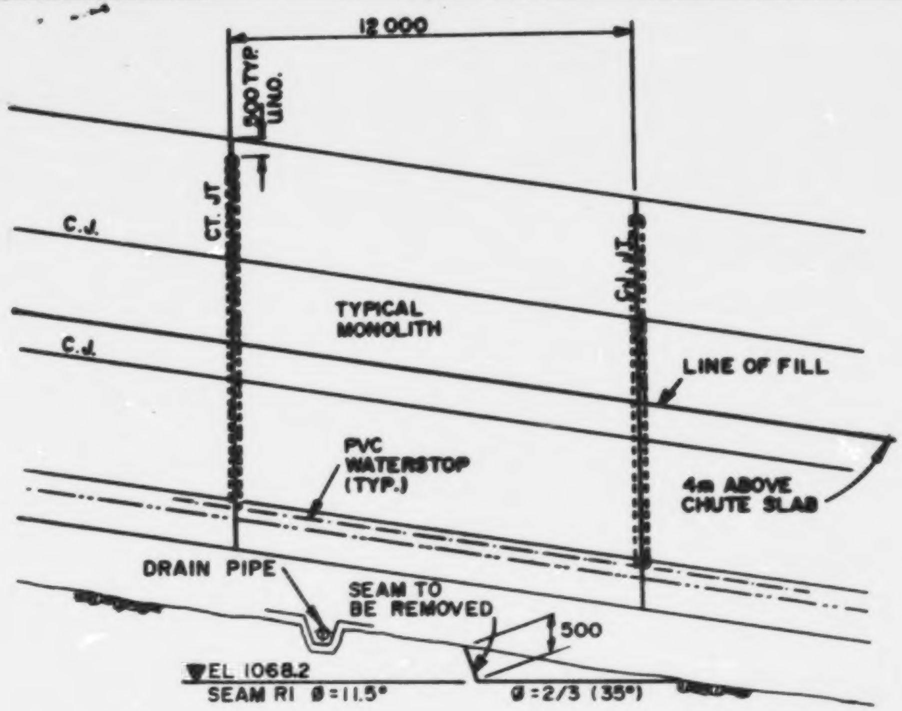

400
1900
16 200
LINE OF FILL ELEV. VARIES
10 500
C.J.
C.J.
C.J.
600 CHUTE SLAB
100 CONC. MUD SLAB
2400 ZONE 3 FILL
200 ZONE 2 FILL
1200
FILL CONCRETE
18 500
UPSTREAM FACE
ASSUMED Ø = 17.5 (TYP.)

SECTION - MONOLITH 2L

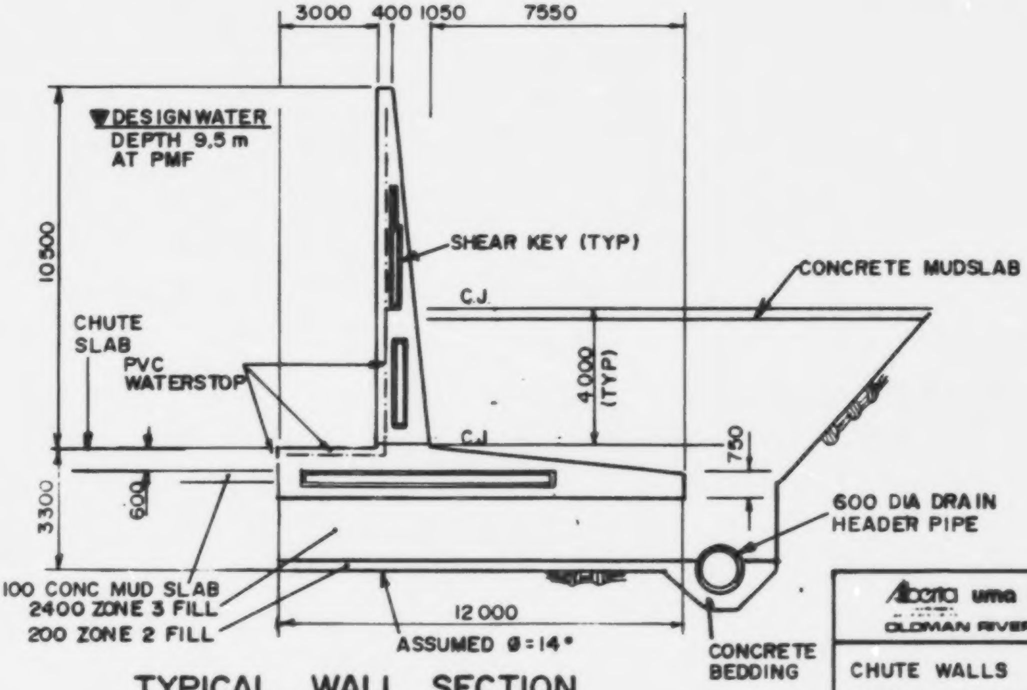
Alberta
OLDMAN RIVER DAM

CHUTE WALLS
MONOLITHS 1, 2 AND 3

PLATE 23


12 000
500 TYP. U.N.O.
CT. JT.
C.J.
CT. JT.
TYPICAL MONOLITH
C.J.
LINE OF FILL
PVC WATERSTOP (TYP.)
4m ABOVE CHUTE SLAB
DRAIN PIPE
SEAM TO BE REMOVED
500
▼EL 1068.2
SEAM R1 Ø=11.5°
Ø=2/3 (35°)

TYPICAL WALL ELEVATION

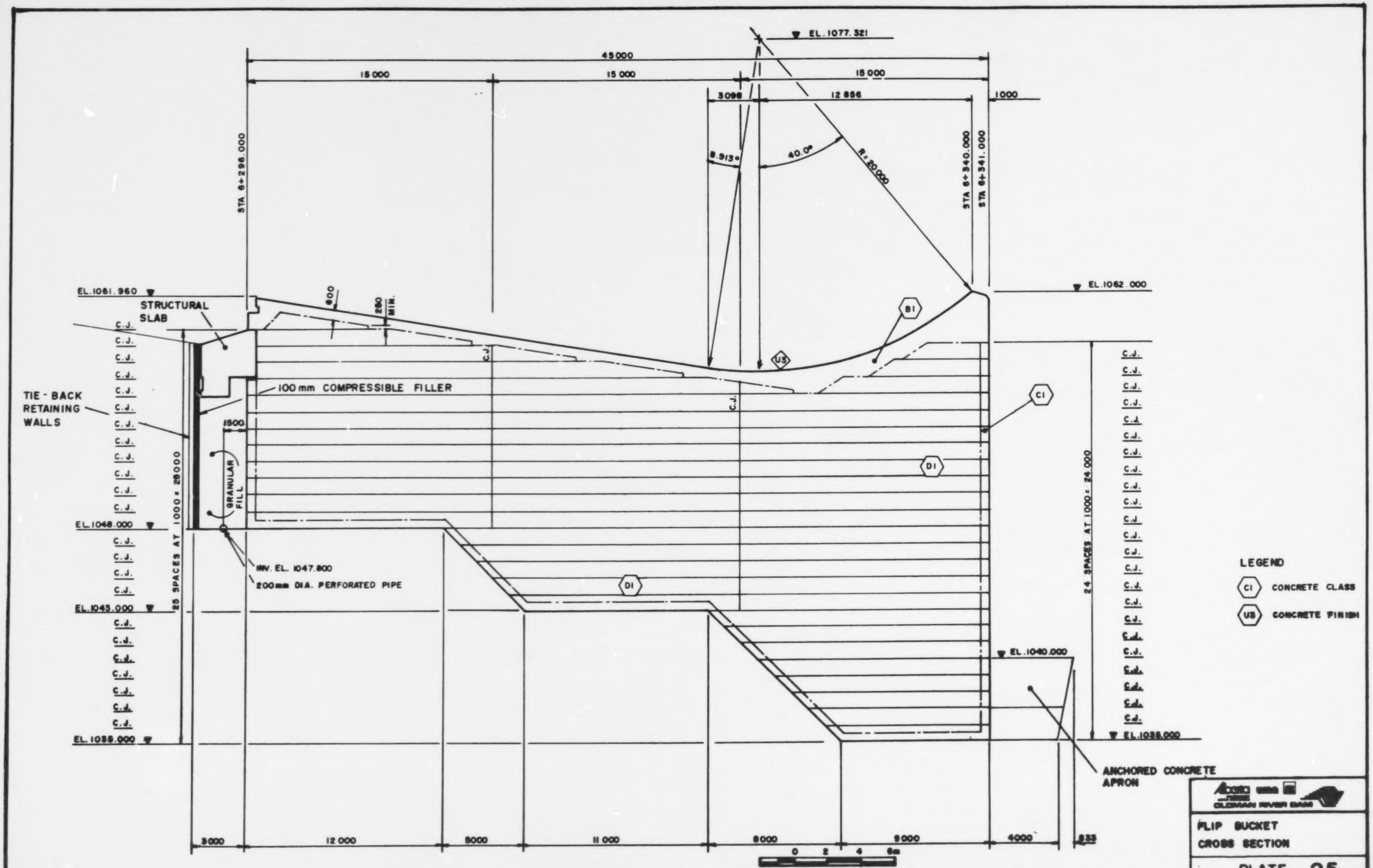

3000
400
1050
7550
▼DESIGN WATER DEPTH 9.5 m AT PMF
10500
SHEAR KEY (TYP)
CONCRETE MUDSLAB
C.J.
CHUTE SLAB
PVC WATERSTOP
4000 (TYP)
C.J.
750
3300
600
600 DIA DRAIN HEADER PIPE
100 CONC MUD SLAB
2400 ZONE 3 FILL
200 ZONE 2 FILL
12 000
ASSUMED Ø=14°
CONCRETE BEDDING

TYPICAL WALL SECTION

OLDMAN RIVER DAM

CHUTE WALLS
MONOLITHS 4 TO 23

PLATE 24

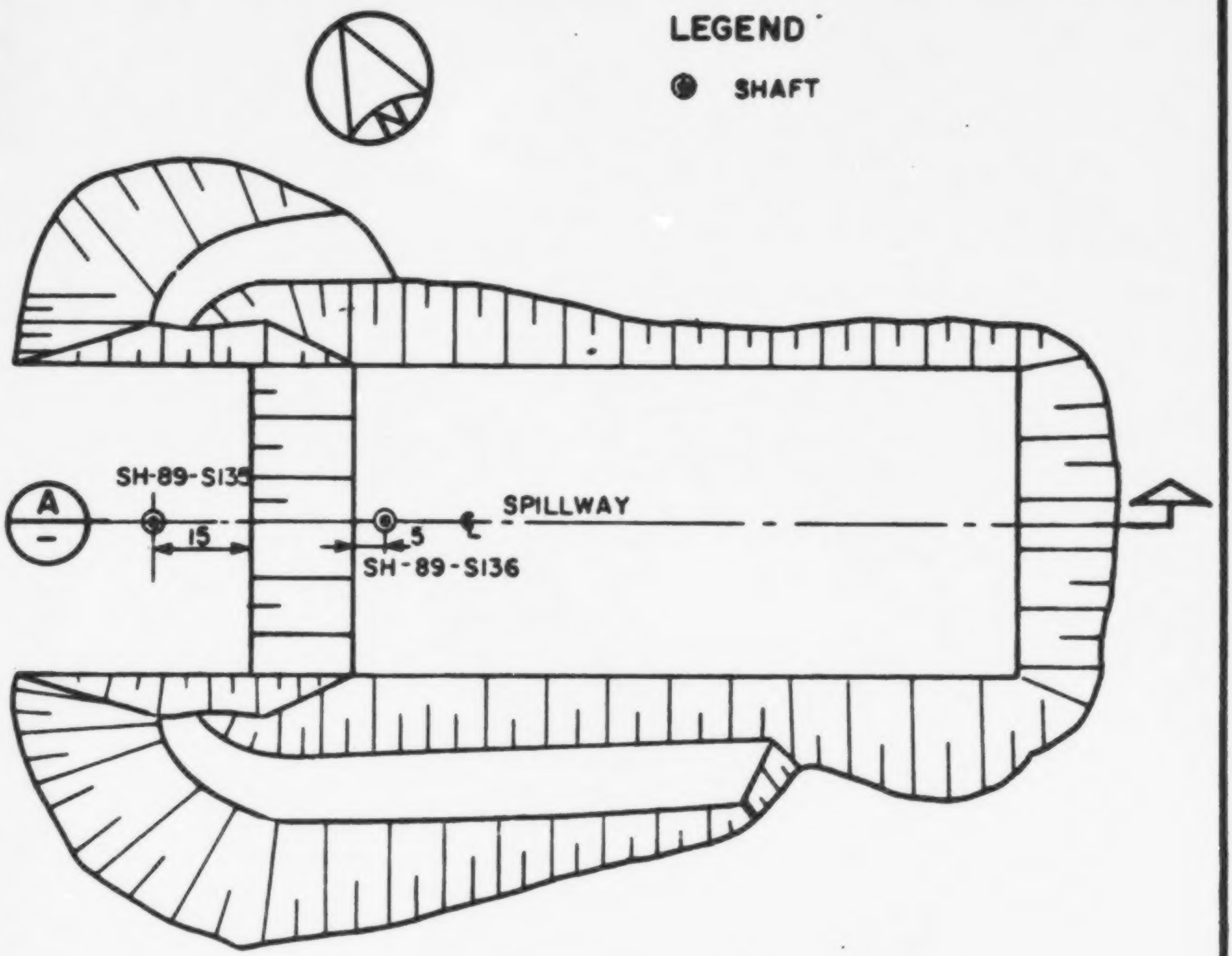

EL. 1077.321
45000
15 000
15 000
15 000
3098
12 856
1000
STA 6+296.000
STA 6+340.000
STA 6+341.000
8.913°
40.0°
R=20.000
EL.1061.960
EL.1062.000
STRUCTURAL SLAB
600
250 MIN.
B1
U3
C1
D1
D1
C.J.
100 mm COMPRESSIBLE FILLER
TIE - BACK RETAINING WALLS
1500
GRANULAR FILL
EL.1048.000
INV. EL. 1047.800
200mm DIA. PERFORATED PIPE
EL.1043.000
EL.1040.000
EL.1038.000
EL.1038.000
25 SPACES AT 1000 = 25000
24 SPACES AT 1000 = 24 000
ANCHORED CONCRETE APRON
3000
12 000
5000
11 000
8000
9000
4000
933
0 2 4 6m
2 1
LEGEND
C1 CONCRETE CLASS
U3 CONCRETE FINISH

OLDMAN RIVER DAM

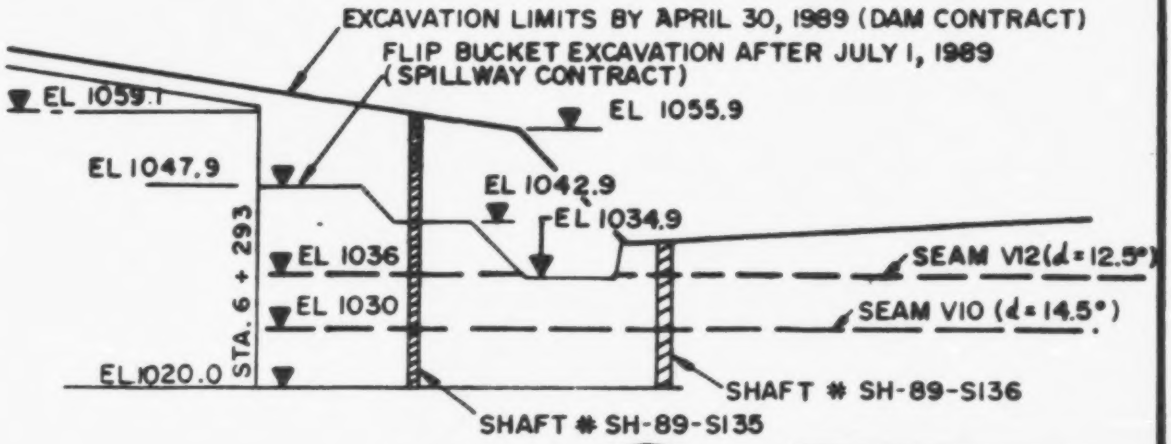
FLIP BUCKET

CROSS SECTION

PLATE 25


LEGEND
SHAFT
N
SH-89-S135
SH-89-S136
15
5
SPILLWAY
A

PLAN
SCALE 1:1000

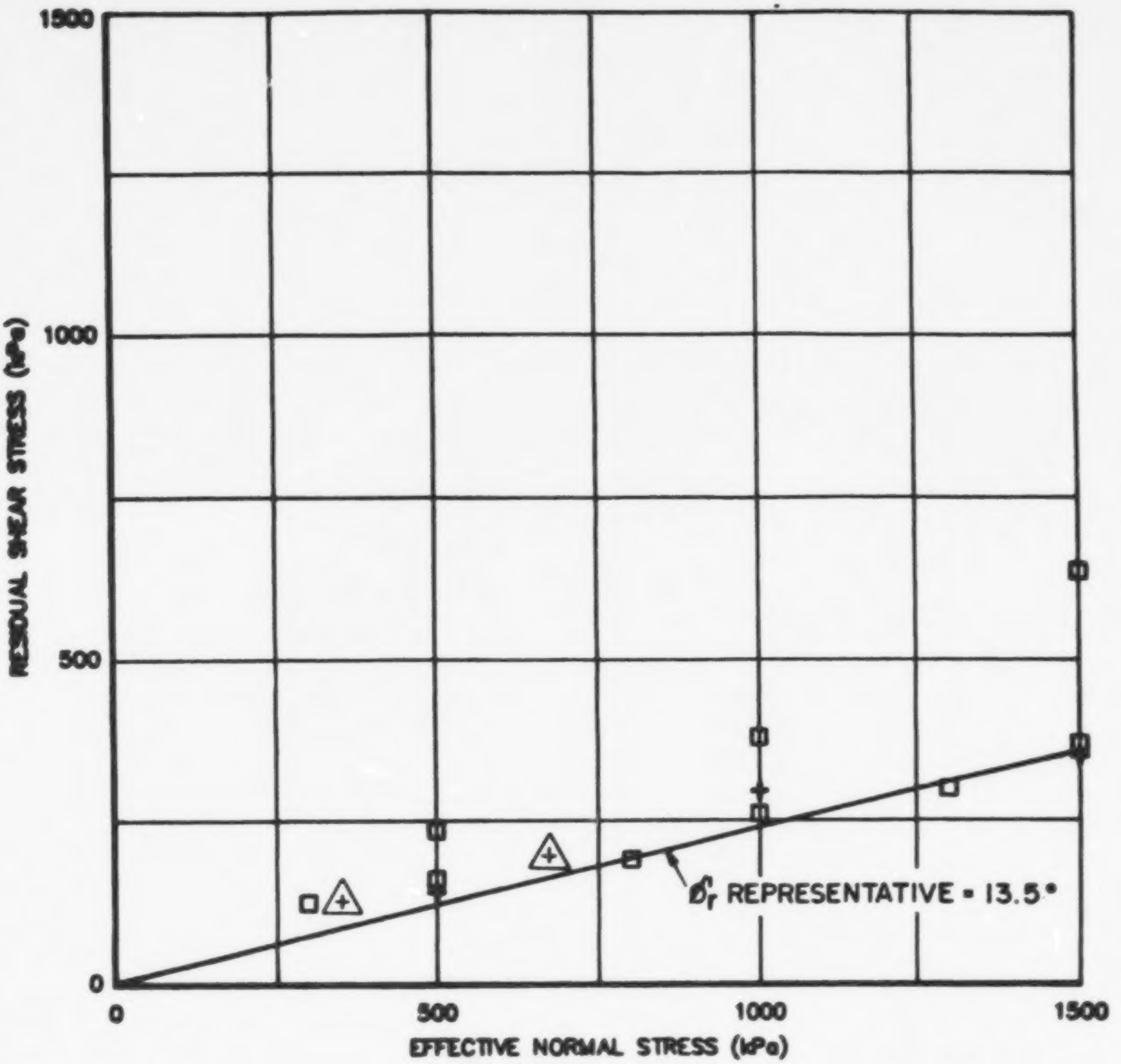

EXCAVATION LIMITS BY APRIL 30, 1989 (DAM CONTRACT)
FLIP BUCKET EXCAVATION AFTER JULY 1, 1989
(SPILLWAY CONTRACT)
EL 1059.1
EL 1055.9
EL 1047.9
EL 1042.9
EL 1034.9
EL 1036
EL 1030
EL1020.0
STA. 6 + 293
SEAM V12(d=12.5°)
SEAM V10 (d=14.5°)
SHAFT # SH-89-S136
SHAFT # SH-89-S135

SECTION A
SCALE 1:1000

OLDMAN RIVER DAM

FLIP BUCKET INVESTIGATION

PLATE 26



SHEAR STRENGTH – RESIDUAL
SEAM NO. V10
ELEVATION = 1030m

LEGEND:

● – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING
5 cm x 5 cm DIRECT SHEAR TEST

☐ – SEAM SAMPLE OBTAINED FROM DRILLHOLE
5 cm x 5 cm DIRECT SHEAR TEST

+ – SEAM SAMPLE OBTAINED FROM SHAFT,
10cm x 10cm DIRECT SHEAR TEST.

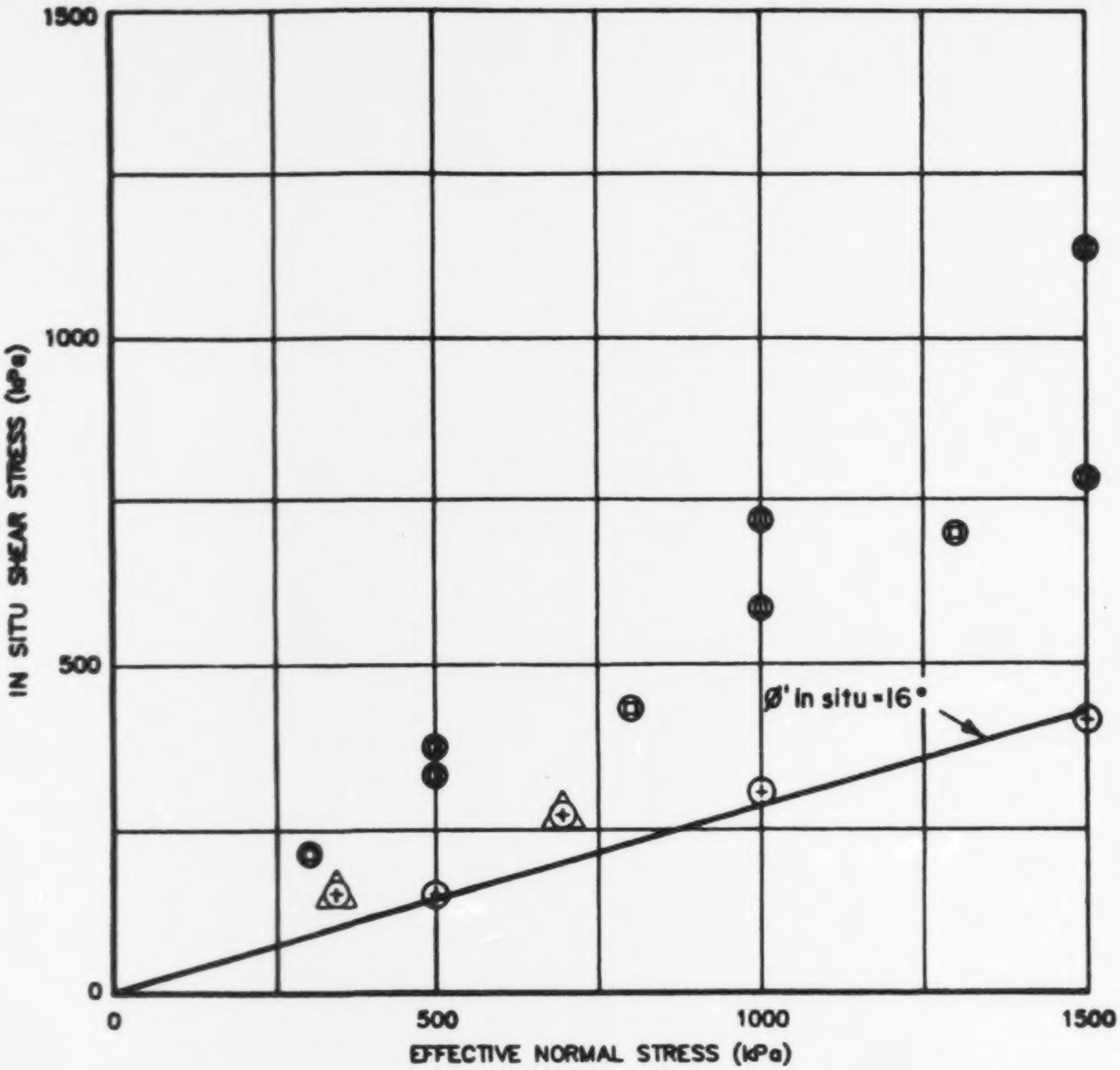
1989 FLIP BUCKET FOUNDATION
INVESTIGATION SHAFTS
SH-89-S135
SH-89-S136

△ – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING, 5cm x 5cm DIRECT SHEAR TEST.

△ – SEAM SAMPLE OBTAINED FROM SHAFT, 10cm x 10cm DIRECT SHEAR TEST.

SHEAR STRENGTH-RESIDUAL
SEAM V10

PLATE 27



SHEAR STRENGTH – IN SITU
SEAM NO. V10
ELEVATION = 1030 m

LEGEND:

● – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING
5 cm x 5 cm DIRECT SHEAR TEST

◎ – SEAM SAMPLE OBTAINED FROM DRILLHOLE
5 cm x 5 cm DIRECT SHEAR TEST

⊕ – SEAM SAMPLE OBTAINED FROM SHAFT,
10 cm x 10 cm DIRECT SHEAR TEST.

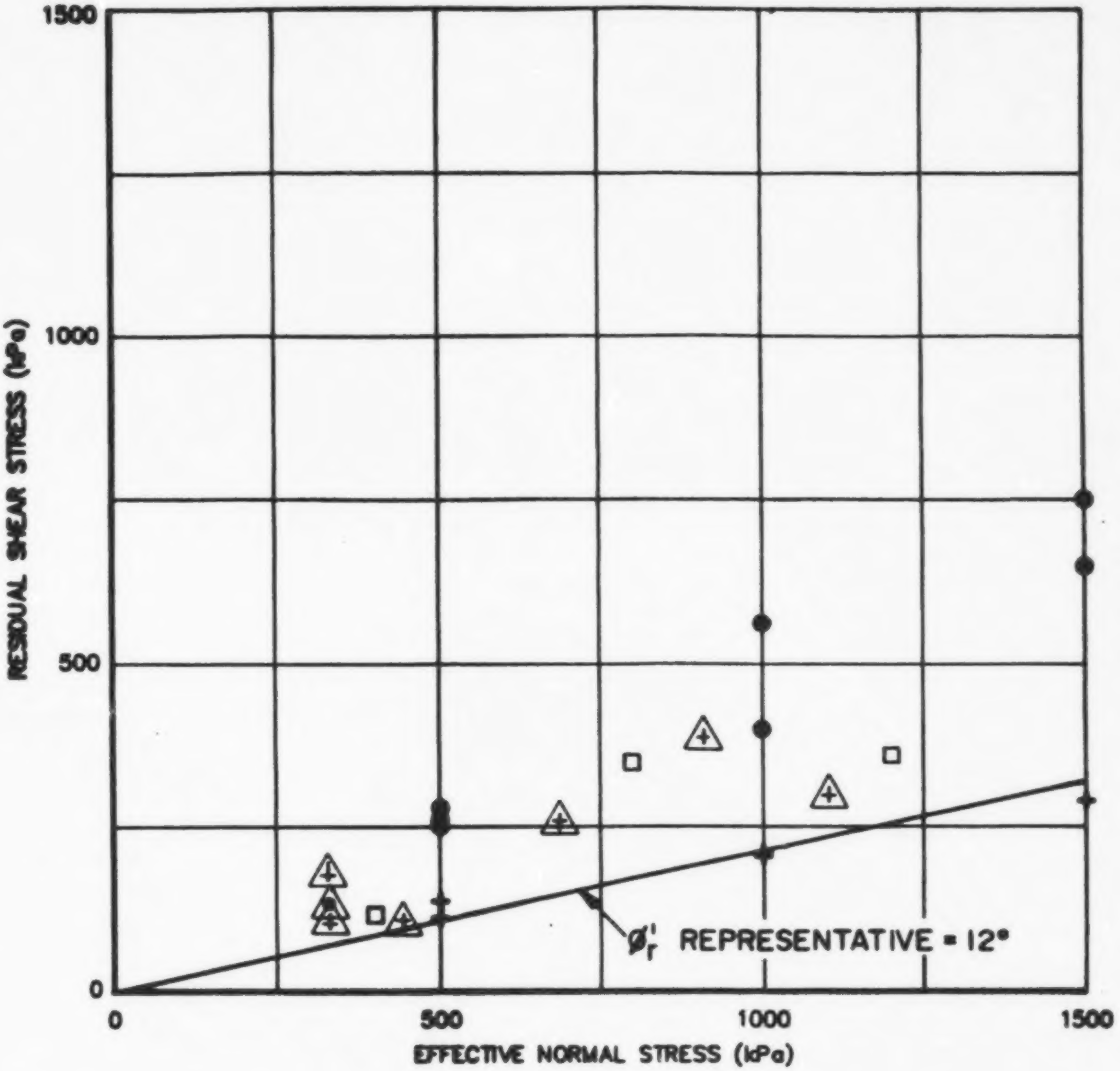
1989 FLIP BUCKET FOUNDATION
INVESTIGATION SHAFTS
SH-89-S135
SH-89-S136

△ — SEAM SAMPLE OBTAINED FROM SHAFT,
10cm x 10cm DIRECT SHEAR TEST.

OLDMAN RIVER DAM

SHEAR STRENGTH – IN SITU
SEAM V10

PLATE 28



SHEAR STRENGTH – RESIDUAL
SEAM No. V12
ELEVATION=1305m

LEGEND:

● – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING
5 cm x 5 cm DIRECT SHEAR TEST

□ – SEAM SAMPLE OBTAINED FROM DRILLHOLE
5 cm x 5 cm DIRECT SHEAR TEST

+ – SEAM SAMPLE OBTAINED FROM SHAFT
10cm x 10cm DIRECT SHEAR TEST.

1989 FLIP BUCKET FOUNDATION
INVESTIGATION SHAFTS
SH-89-S135
SH-89-S136

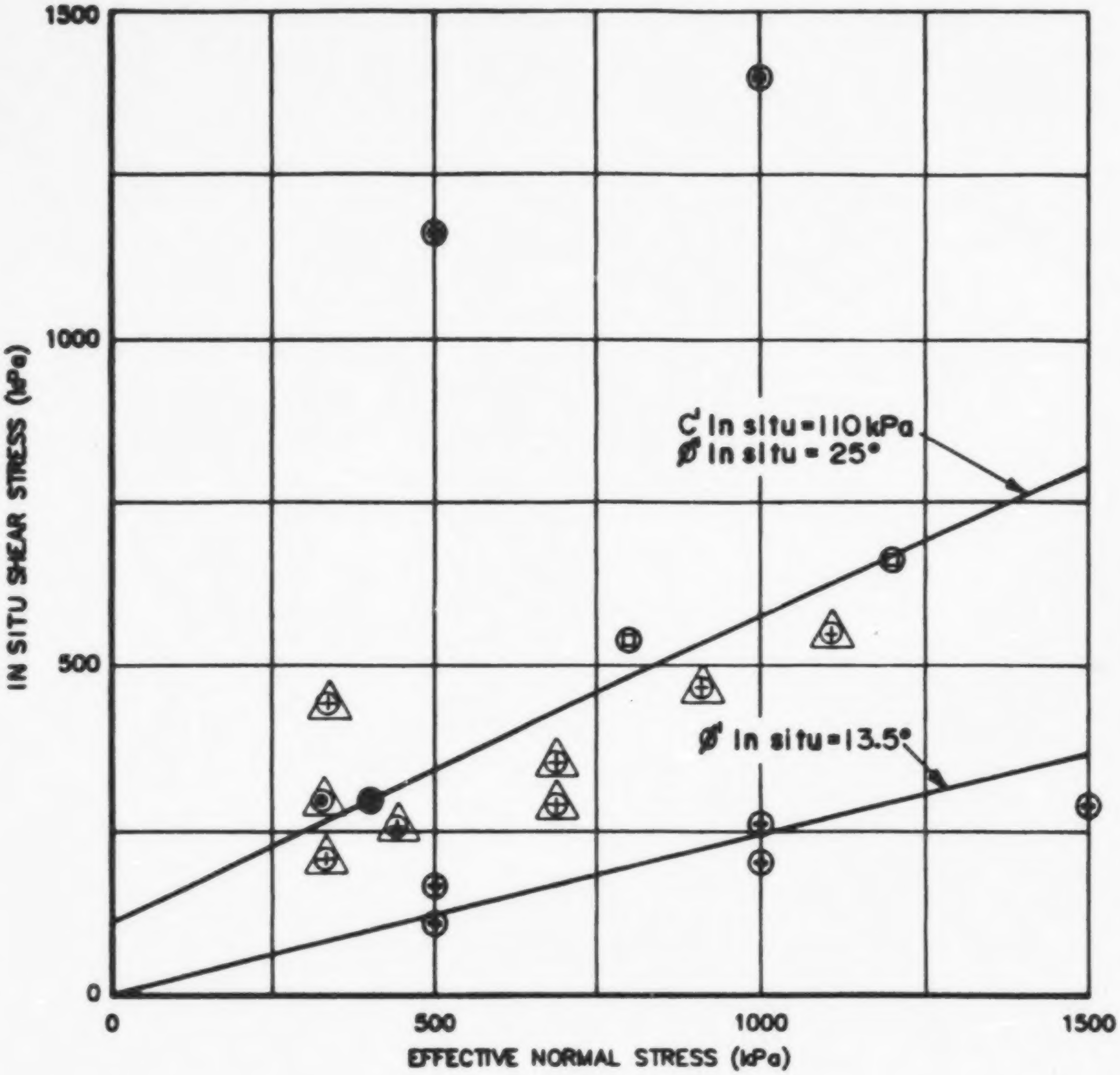
△● – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING, 5cm x 5cm DIRECT SHEAR TEST.

△+ – SEAM SAMPLE OBTAINED FROM SHAFT, 10cm x 10cm DIRECT SHEAR TEST.

OLDMAN RIVER DAM

SHEAR STRENGTH-RESIDUAL
SEAM V12

PLATE 29



SHEAR STRENGTH – IN SITU
SEAM No V12
ELEVATION ± 1035 m

LEGEND:

● – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING 5 cm x 5 cm DIRECT SHEAR TEST

⊙ – SEAM SAMPLE OBTAINED FROM DRILLHOLE 5 cm x 5 cm DIRECT SHEAR TEST

⊕ – SEAM SAMPLE OBTAINED FROM SHAFT, 10cm x 10cm DIRECT SHEAR TEST

1989 FLIP BUCKET FOUNDATION
INVESTIGATION SHAFTS
SH-89-S135
SH-89-S136

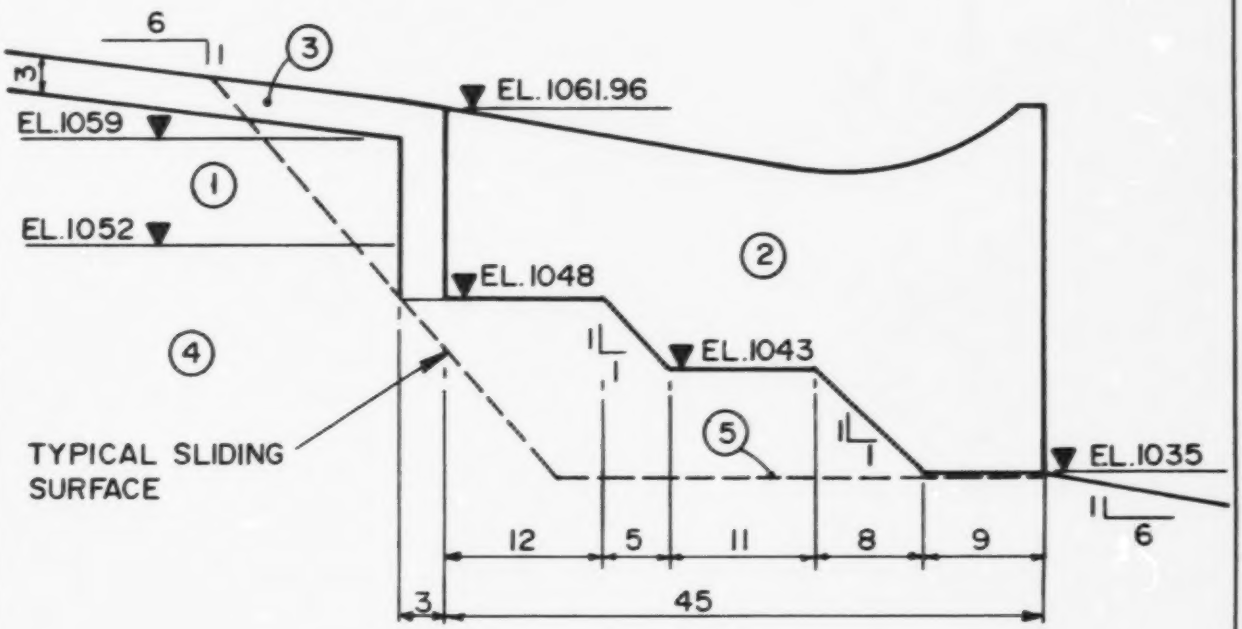
△⊙ – INTACT CLAYSTONE SAMPLE SHEARED ALONG BEDDING, 5cm x 5cm DIRECT SHEAR TEST.

△⊕ – SEAM SAMPLE OBTAINED FROM SHAFT, 10cm x 10cm DIRECT SHEAR TEST.

Alberta
OLDMAN RIVER DAM

SHEAR STRENGTH – IN SITU
SEAM V12

PLATE 30


6
1
3
③
EL. 1061.96
EL.1059
①
EL.1052
②
EL.1048
④
1
1
EL.1043
TYPICAL SLIDING SURFACE
⑤
1
1
EL.1035
1
6
12
5
11
8
9
3
45

CROSS-SECTION

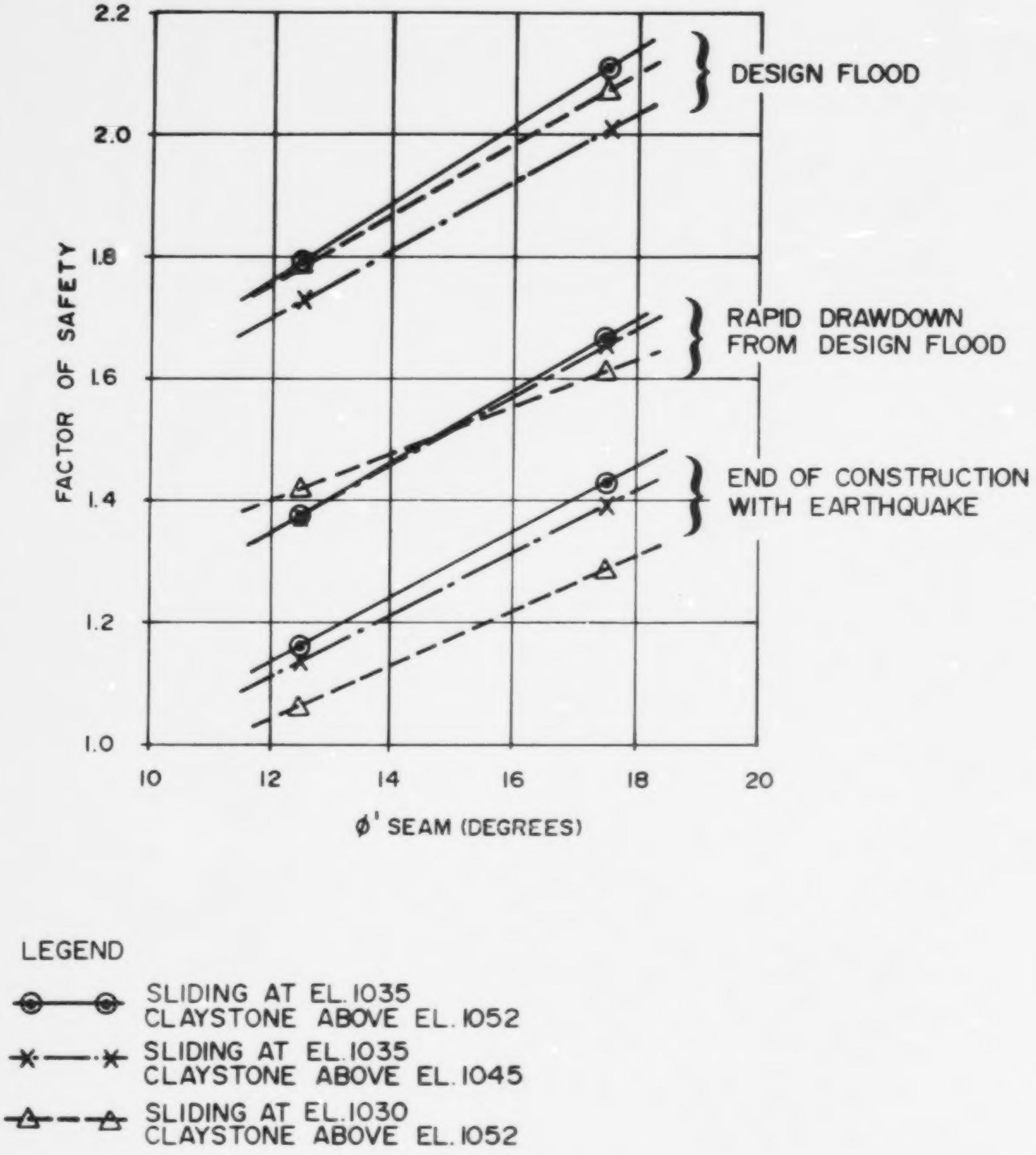
MATERIAL PROPERTIES

①	CLAYSTONE	$c'_{x\text{-bed}} = 50$ kPa	$\phi'_{x\text{-bed}} = 35°$	$\gamma = 23.6$ kN/m^3
②	CONCRETE	$c' = 1000$ kPa	$\phi' = 60°$	$\gamma = 22.2$ kN/m^{3*}
③	GRAVEL	$c' = 0$	$\phi' = 35°$	$\gamma = 21.6$ kN/m^3
④	SANDSTONE	$c' = 0$	$\phi'_{x\text{-bed}} = 60°$	$\gamma = 23.6$ kN/m^3
⑤	ALONG SEAM	$c' = 0$	$\phi' = 12.5°$	$\gamma = 23.6$ kN/m^3

*CONCRETE UNIT WEIGHT ADJUSTED TO CONSIDER VOIDS DUE TO PRESENCE OF GALLERIES.

FLIP BUCKET
STABILITY

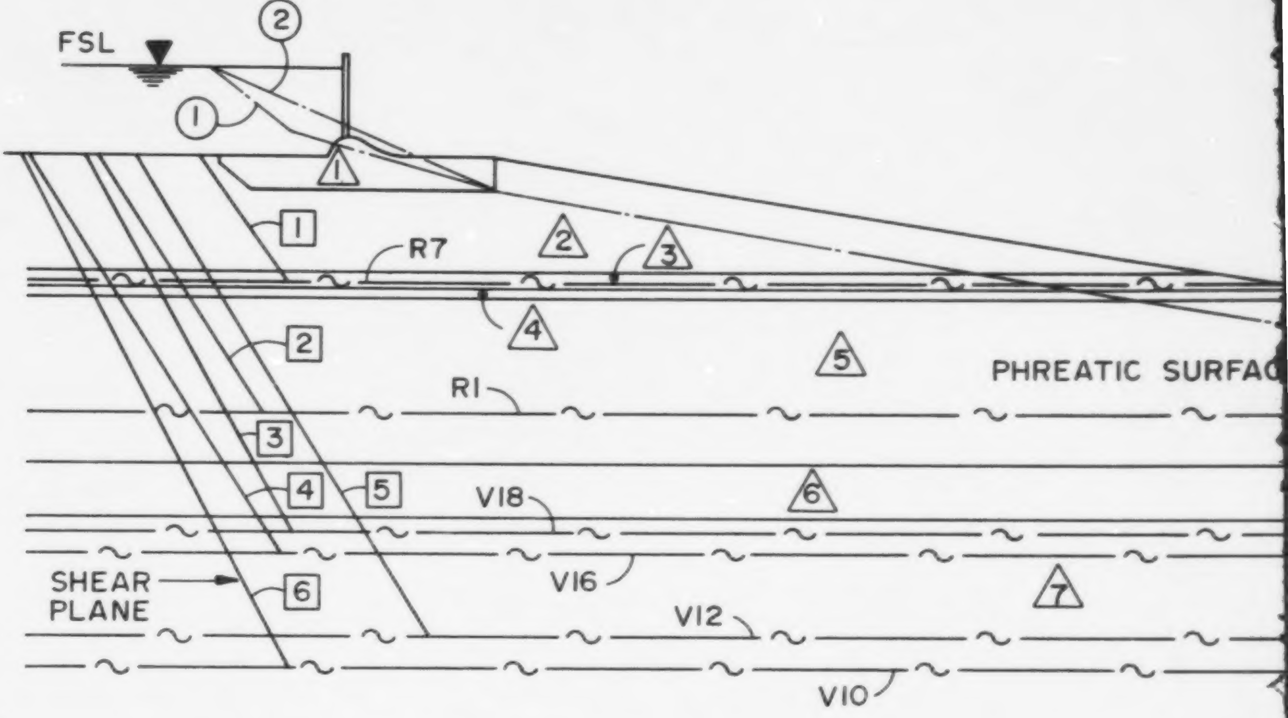
PLATE 31



Alberta
OLDMAN RIVER DAM

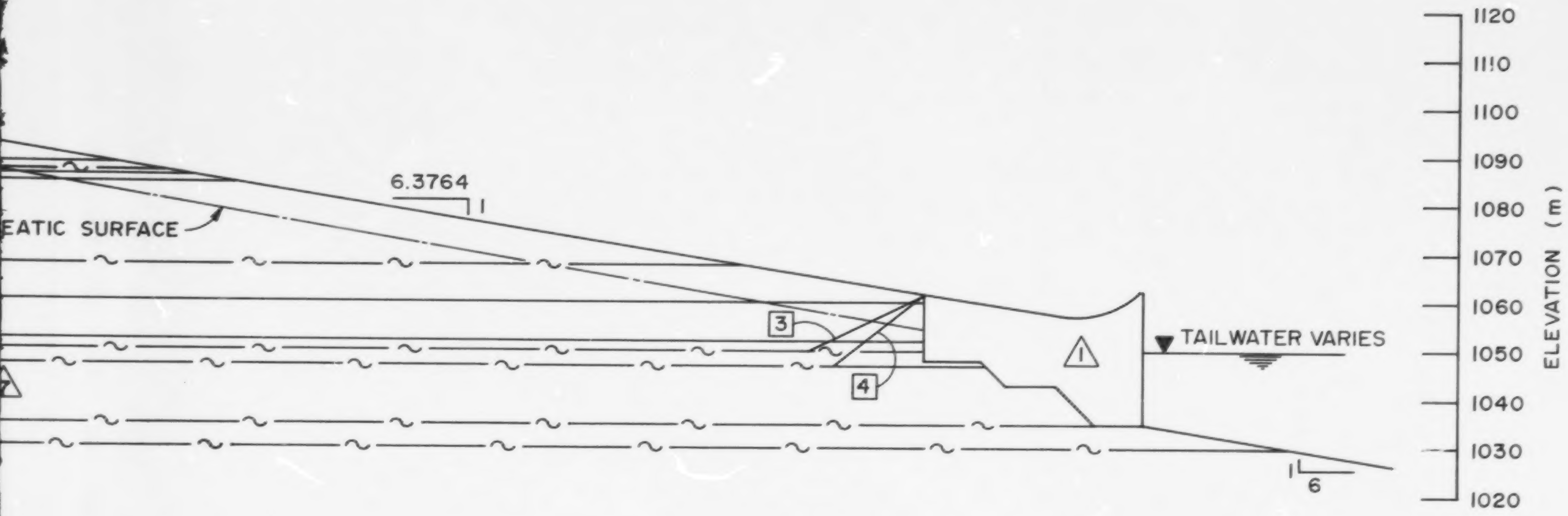
FLIP BUCKET
STABILITY
SENSITIVITY ANALYSIS

PLATE **32**


FSL
②
①
1
1
R7
2
3
4
2
5
PHREATIC SURFAC
R1
3
4
5
V18
6
SHEAR PLANE
6
V16
7
V12
V10

CASE \ SEAM	R7, EL. 1086.5		R1, EL. 1068.2		V18
	F.S.	SHEAR PLANE	F.S.	SHEAR PLANE	F.S.
1. RESERVOIR AT FSL, GATES CLOSED, GROUT CURTAIN AND DRAINS EFFECTIVE (PHREATIC SURFACE NO. 1)	1.6	$\boxed{1}$	1.6	$\boxed{2}$	2.1
2. RESERVOIR AT FSL, GATES CLOSED, INEFFECTIVE GROUT CURTAIN, DRAINS BLOCKED (PHREATIC SURFACE NO. 2)	1.5	$\boxed{1}$	1.6	$\boxed{2}$	2.1
3. CASE 1 WITH HORIZONTAL SEISMIC LOADING OF 0.065g	1.1	$\boxed{1}$	1.1	$\boxed{2}$	1.5

FACTORS OF SAFETY AND SHEAR PLANES (2-D STAB


6.3764
1
EATIC SURFACE
3
4
1
TAILWATER VARIES
1
6
1120
1110
1100
1090
1080
1070
1060
1050
1040
1030
1020
ELEVATION (m)
7

1068.2	V18, EL.1049.4		V16, EL.1046.8		V12, EL.1035		V10, EL.1030		MINIMUM REQUIRED FACTOR OF SAFETY
SHEAR PLANE	F.S.	SHEAR PLANE	F.S.	SHEAR PLANE	F.S.	SHEAR PLANE	F.S.	SHEAR PLANE	
2	2.1	3	2.4	4	1.6	5	1.8	6	1.5
2	2.1	3	2.3	4	1.6	5	1.8	6	1.3
2	1.5	3	1.7	4	1.1	5	1.2	6	>1.0

(2-D STABILITY ANALYSIS)

SEAM	SHEAR STRENGTH	
	C' (kPa)	ϕ' (DEG.)
R7	0	13
R1	0	11.5
V18	0	16
V16	0	17
V12	0	12.5
V10	0	14.5

	MATERIAL
1	CONCRETE
2	SLIGHTLY WE MUDROCK
3	SLIGHTLY WE MUDROCK
4	SANDSTONE
5	INTACT MUDR
6	SLIGHTLY WE MUDROCK
7	SANDSTONE

LEGEND:

① HEAD DROPS FROM FSL AT HEEL TO
[TWL + 1/3 (FSL - TWL)]
AT DRAINS AND TO TWL AT TOE.

② LINEAR VARIATION FROM FSL AT
HEEL TO TWL AT TOE.

——∿—— SEAM

HEEL
D
TW
FSL

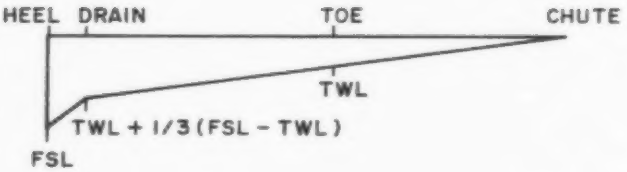
STRENGTH
ϕ' (DEG.)
13
11.5
16
17
12.5
14.5

MATERIAL	CROSS BEDDING SHEAR STRENGTH	
	C' (kPa)	ϕ' (DEG.)
△1 CONCRETE	1000	60
△2 SLIGHTLY WEATHERED MUDROCK	50	35
△3 SLIGHTLY WEATHERED MUDROCK	100	35
△4 SANDSTONE	0	60
△5 INTACT MUDROCK	200	35
△6 SLIGHTLY WEATHERED MUDROCK	50	35
△7 SANDSTONE	0	60

DROPS FROM FSL AT HEEL TO
+ 1/3 (FSL - TWL)]
AINS AND TO TWL AT TOE.

VARIATION FROM FSL AT
TO TWL AT TOE.

—— . . . SEAM

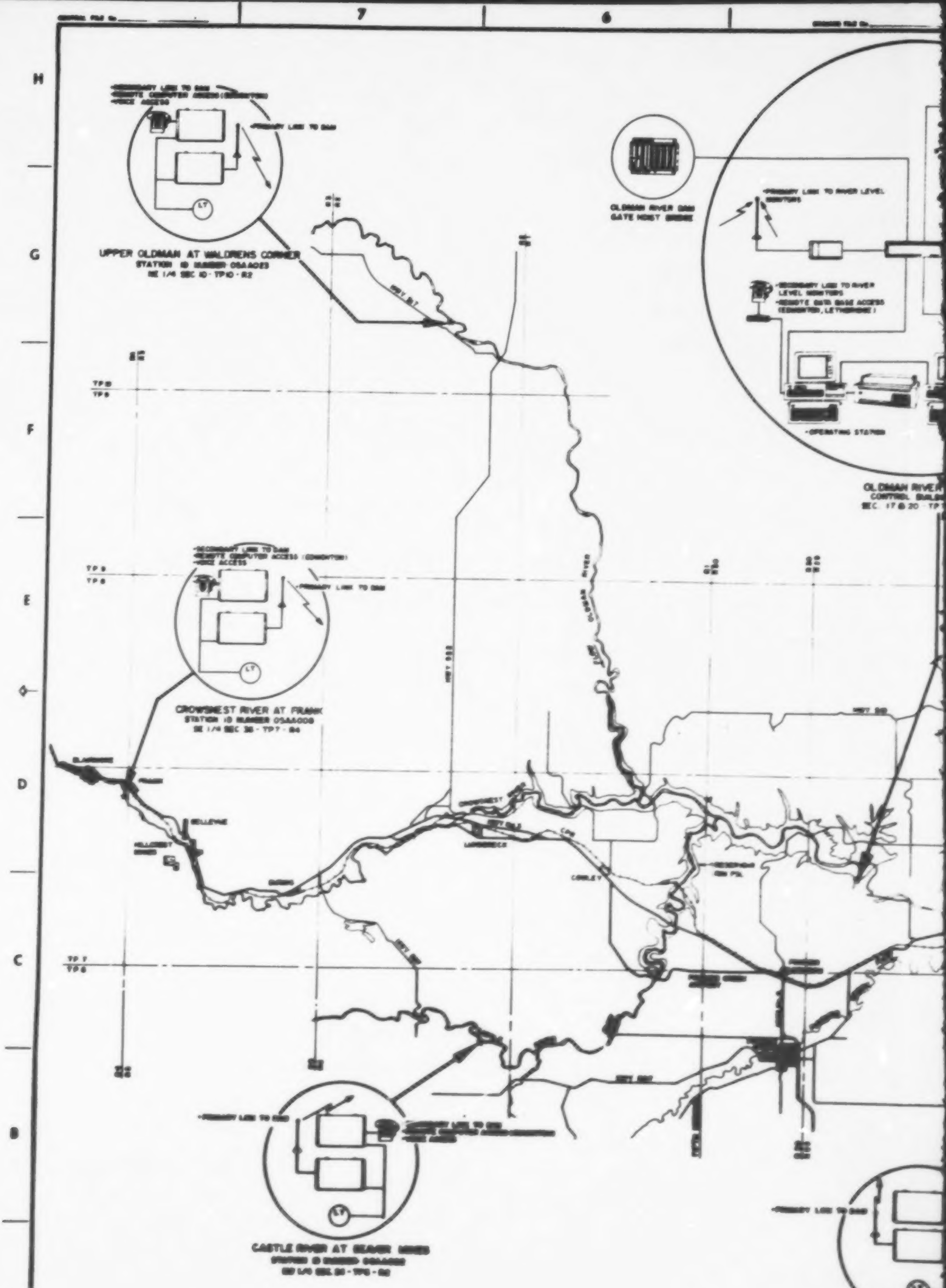


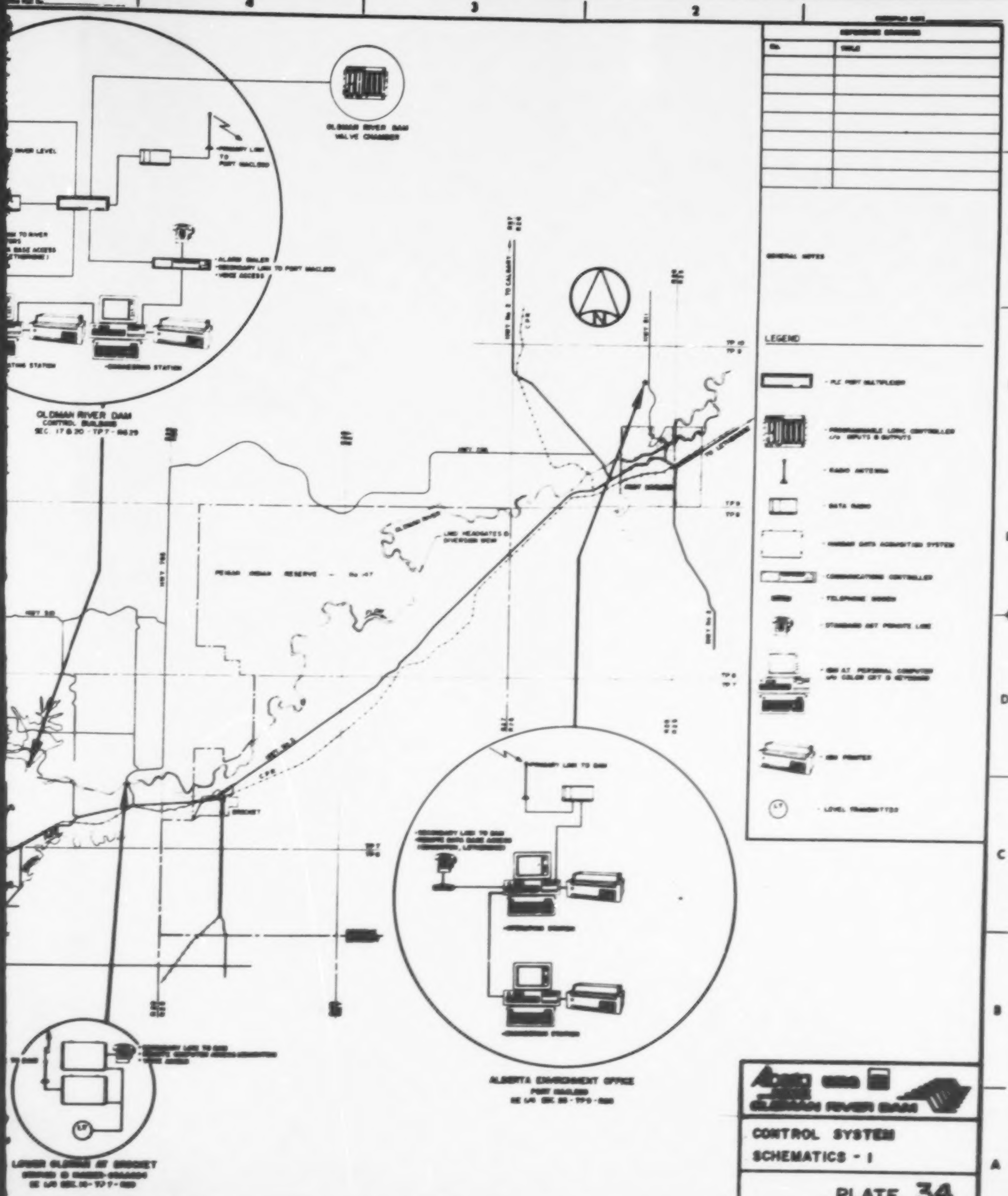
PORE WATER PRESSURES ALONG SHEAR PLANE

Alberta
OLDMAN RIVER DAM

SPILLWAY STABILITY

PLATE 33

7
6
H
G
F
E
D
C
B
A
•SECONDARY LINK TO DAM
•REMOTE COMPUTER ACCESS
•VOICE ACCESS
•PRIMARY LINK TO DAM
LT
UPPER OLDMAN AT WALDRENS CORNER
STATION ID NUMBER 05AA023
SE 1/4 SEC 10 · TP10 · R2
OLDMAN RIVER DAM
GATE HOIST BRIDGE
•PRIMARY LINK TO RIVER LEVEL MONITORS
•SECONDARY LINK TO RIVER LEVEL MONITORS
•REMOTE DATA BASE ACCESS (EDMONTON, LETHBRIDGE)
•OPERATING STATION
OLDMAN RIVER
CONTROL BUILD
SEC. 17 & 20 · TP
TP 10
TP 9
TP 8
TP 7
TP 6
HWY 22
OLDMAN RIVER
•SECONDARY LINK TO DAM
•REMOTE COMPUTER ACCESS (EDMONTON)
•VOICE ACCESS
•PRIMARY LINK TO DAM
LT
CROWSNEST RIVER AT FRANK
STATION ID NUMBER 05AA008
SE 1/4 SEC 36 · TP7 · R4
HWY 3
FRANK
BELLEVUE
HILLCREST
LUNDBRECK
COWLEY
•PRIMARY LINK TO DAM
LT
CASTLE RIVER AT BEAVER MINES
•PRIMARY LINK TO DAM
LT


4
3
2
H
G
F
E
D
C
B
A
REFERENCE DRAWINGS
GENERAL NOTES
LEGEND
PLC PORT MULTIPLEXER
PROGRAMMABLE LOGIC CONTROLLER c/w INPUTS & OUTPUTS
RADIO ANTENNA
DATA RADIO
DATA ACQUISITION SYSTEM
COMMUNICATIONS CONTROLLER
TELEPHONE MODEM
IBM AT PERSONAL COMPUTER c/w COLOR CRT & KEYBOARD
IBM PRINTER
LEVEL TRANSMITTER
LT
OLDMAN RIVER DAM
VALVE CHAMBER
RIVER LEVEL
PRIMARY LINK TO FORT MACLEOD
ALARM DIALER
SECONDARY LINK TO FORT MACLEOD
VOICE ACCESS
ENGINEERING STATION
OLDMAN RIVER DAM
CONTROL BUILDING
SEC. 17 & 20 - TP 7 - R 29
N
HWY 3
HWY 2 TO CALGARY
HWY 811
TP 10
TP 9
TP 8
TP 7
FORT MACLEOD
TO LETHBRIDGE
OLDMAN RIVER
PEIGAN INDIAN RESERVE
CPR
BROCKET
PRIMARY LINK TO DAM
SECONDARY LINK TO DAM
ALBERTA ENVIRONMENT OFFICE
FORT MACLEOD
LOWER OLDMAN AT BROCKET

Alberta
OLDMAN RIVER DAM

CONTROL SYSTEM
SCHEMATICS - I

PLATE 34

H

RESERVOIR LEVEL

[illegible]

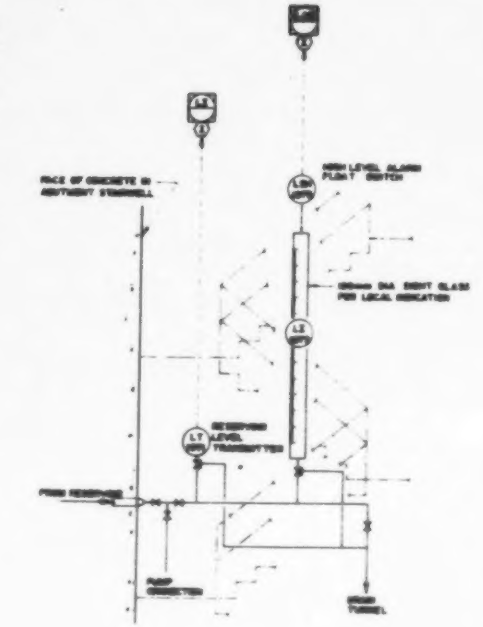
[illegible]

GRADUATED SIGHT GLASS PROVIDES ACCURATE LOCAL INDICATION, INDEPENDENT OF ELECTRONICS

[illegible] (SEE DETAIL 3).

G

F

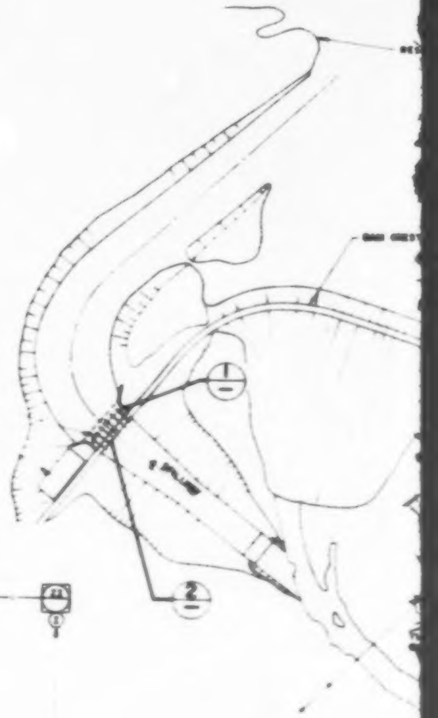


FACE OF CONCRETE IN ABUTMENT STAIRWELL
HIGH LEVEL ALARM FLOAT SWITCH
100mm DIA. SIGHT GLASS FOR LOCAL INDICATION
LI
LT
RESERVOIR LEVEL TRANSMITTER
FROM RESERVOIR
PUMP CONNECTION
DRAIN TUNNEL

DETAIL ① - TYPICAL RESERVOIR LEVEL INSTRUMENTATION

(STAIRWELL 2 SHOWN)

E



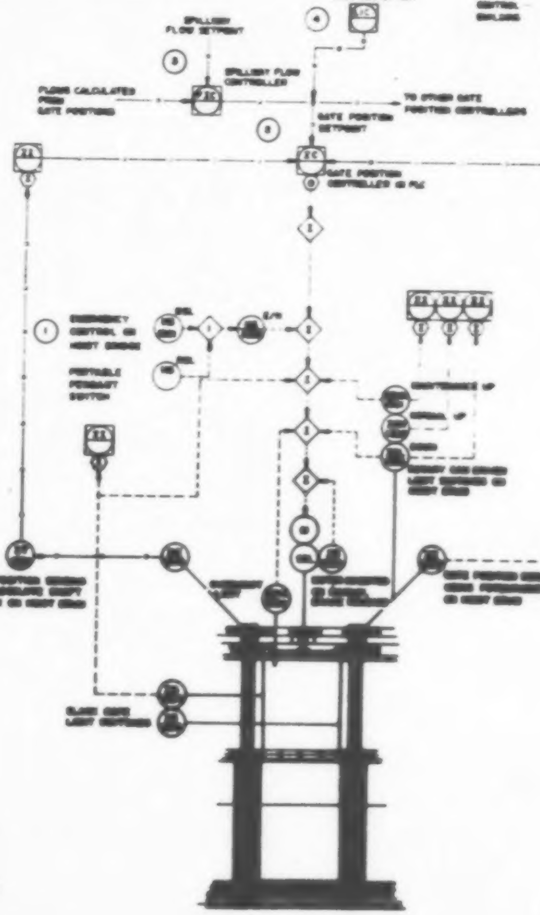
SPILLWAY GATES
CONTROL BUILDING
1
2
SITE PL

SPILLWAY GATE OPERATION

① LOCAL EMERGENCY CONTROL

- PLC BYPASSED
- CONTROL BUILDING BYPASSED
- HS-1000 IN EMERGENCY
- OPERATOR MOVES GATE USING KEYSWITCH (HS-1001) OR PORTABLE PENDANT SWITCH (FOR MAINTENANCE)
- DIGITAL INDICATOR (ZI-1000) PROVIDES POSITION FEEDBACK

D



AUTOMATIC DAM PROTECTION
SPILLWAY FLOW SETPOINT
SPILLWAY FLOW CONTROLLER
FLOW CALCULATED FROM GATE POSITIONS
TO OTHER GATE POSITION CONTROLLERS
GATE POSITION SETPOINT
GATE POSITION CONTROLLER IN PLC
EMERGENCY CONTROL IN HOIST HOUSE
PORTABLE PENDANT SWITCH
MAINTENANCE UP
NORMAL UP
DOWN
GATE POSITION SENSOR
SLACK GATE LIGHT SWITCHES

DIGITAL POSITION INDICATOR IN HOIST HOUSE

C

② GATE POSITION CONTROL

- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]

B

③ SPILLWAY FLOW CONTROL

- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]
- [illegible]

④ AUTOMATIC DAM PROTECTION

- [illegible]
- [illegible]
- [illegible]
- [illegible]

A

DETAIL ② - TYPICAL GATE INSTRUMENTATION

REFERENCE DRAWINGS

No.	TITLE

RESERVOIR [illegible] (FSL)
TUNNEL INLET
DAM CREST
③
DIVERSION TUNNELS
TUNNEL OUTLET

SITE PLAN

REGULATING VALVES OPERATION

① EMERGENCY CONTROL

- LOCAL ONLY
- PLC BYPASSED
- HS-22V2 IN EMERGENCY
- OPERATOR MOVES VALVE USING KEYSWITCH (HS-22V)
- DIGITAL INDICATOR (ZI-22V) PROVIDES POSITION FEEDBACK

② VALVE POSITION CONTROL

- THROUGH PLC AND OPERATOR CONTROL
- HS-22V2 IN NORMAL
- OPERATOR SETS VALVE POSITION SETPOINT VIA KEYBOARD
- OPERATOR ISSUES "PROCEED" COMMAND
- VALVE POSITION CONTROLLER IN PLC MOVES VALVE TO SETPOINT
- NO FURTHER CHANGES ARE MADE UNTIL VALVE POSITION HAS BEEN ACHIEVED
- POSITION AND SETPOINT INDICATED ON SCREEN

③ TUNNEL FLOW CONTROL

- THROUGH PLC AND OPERATOR CONTROL
- OPERATOR SETS TUNNEL FLOW SETPOINT VIA KEYBOARD
- OPERATOR ISSUES "PROCEED" COMMAND
- FLOW CONTROLLER IN PLC ADJUSTS VALVE POSITION SETPOINTS TO ACHIEVE FLOW SETPOINT
- VALVE POSITION CONTROLLERS IN PLC MOVE VALVES TO VALVE POSITION SETPOINTS
- NO FURTHER CHANGES ARE MADE UNTIL VALVE POSITION HAS BEEN ACHIEVED
- SETPOINTS, FLOW, AND VALVE POSITION INDICATED ON SCREEN

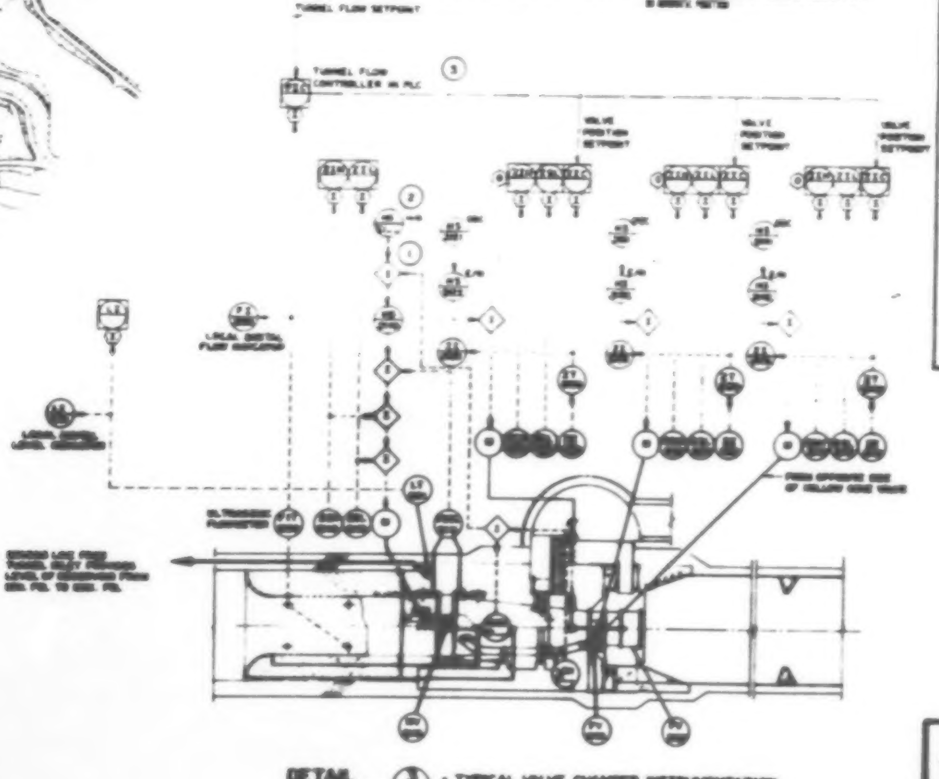
GUARD VALVE OPERATION

HAND CONTROL

- LOCAL CONTROL ONLY
- HAND CONTROL ENABLED BY PLACING KEYSWITCH
- OPERATOR MOVES VALVE USING KEY SWITCH HS-2... IN HAND POSITION
- OPENING PREVENTED BY DIFFERENTIAL ACROSS VALVE TOO HIGH [illegible]

AUTOMATIC CONTROL

- AUTOMATIC VALVE CLOSURE ON CHAMBER FLOOD LEVEL OR HIGH FLOW [illegible]
- [illegible] REGULATING VALVE FAILURE
- VALVE [illegible] IN A [illegible] FULL CLOSED POSITION
- AUTOMATIC CONTROL ENABLED BY PLACING KEY SWITCH HS-2...
IN AUTOMATIC POSITION



TUNNEL FLOW SETPOINT
TUNNEL FLOW CONTROLLER IN PLC
FIC
③
VALVE POSITION SETPOINT
VALVE POSITION SETPOINT
VALVE POSITION SETPOINT
②
①
LOCAL DIGITAL FLOW INDICATOR
LOCAL DIGITAL LEVEL INDICATOR
ULTRASONIC FLOWMETER
FROM OPPOSITE SIDE OF HOLLOW CONE VALVE
SENSING LINE FROM TUNNEL INLET PROVIDES LEVEL OF RESERVOIR FROM [illegible] TO [illegible]
HV
FV
FV

DETAIL ③ - TYPICAL VALVE CHAMBER INSTRUMENTATION

LEGEND

- INSTRUMENTATION SYMBOLOGY FOLLOWS ISA STANDARDS S5.1 & S5.3

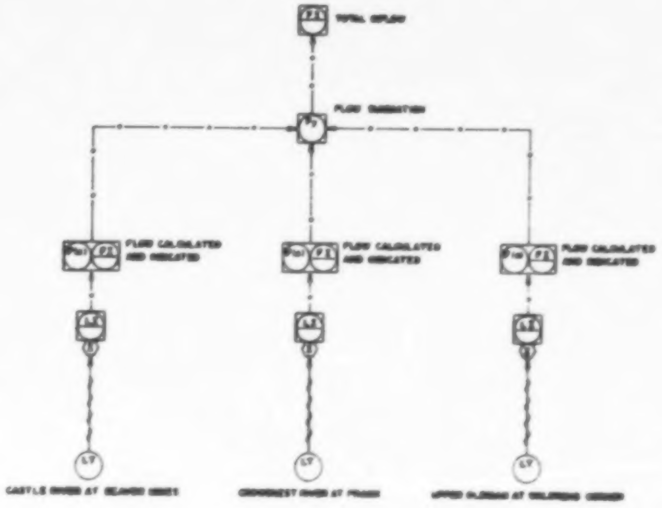
BOX DENOTES SHARED DISPLAY (ON PC)

FI	FLOW INDICATOR	S	SOLENOID
FIC	FLOW INDICATOR / CONTROLLER	ZC	POSITION CONTROLLER
FIT	FLOW INDICATOR / TRANSMITTER	ZE	POSITION ELEMENT
FV	FLOW VALVE	ZE	POSITION ELEMENT
HS E/N	HANDSWITCH EMERGENCY / NORMAL	ZI	POSITION INDICATOR
HS OSC	HANDSWITCH OPEN - STOP - CLOSE	ZI	POSITION INDICATOR
HS RSL	HANDSWITCH RAISE - STOP - LOWER	ZIC	POSITION INDICATOR / CONTROLLER
HV	HAND VALVE	ZIH	POSITION INDICATE HIGH
I	PLC INPUT	ZIL	POSITION INDICATE LOW
I	INTERLOCK	ZS	POSITION SWITCH
LAH	LEVEL ALARM HIGH	ZSH	POSITION SWITCH HIGH
LI	LEVEL INDICATOR	ZSHH	POSITION SWITCH HIGH HIGH
LI	LEVEL INDICATOR	ZSL	POSITION SWITCH LOW
LSH	LEVEL SWITCH HIGH	ZSL	POSITION SWITCH LOW
LT	LEVEL TRANSMITTER	ZT	POSITION TRANSMITTER
M	MOTOR	LIC	LEVEL INDICATOR CONTROLLER
O	PLC OUTPUT		
PDSL	PRESSURE DIFFERENTIAL SWITCH LOW		

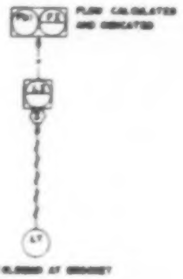
ACRES [illegible]
[illegible] RIVER DAM

CONTROL SYSTEM
SCHEMATICS - 2

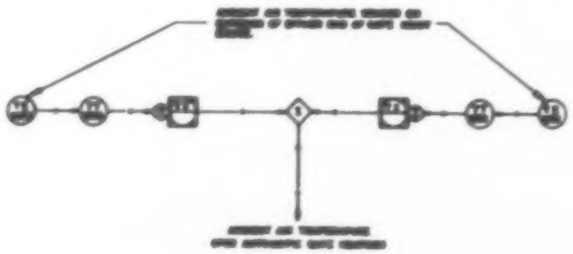
PLATE 35


TOTAL OUTFLOW
FLOW CALCULATED AND INDICATED
FLOW CALCULATED AND INDICATED
FLOW CALCULATED AND INDICATED
LT
LT
LT

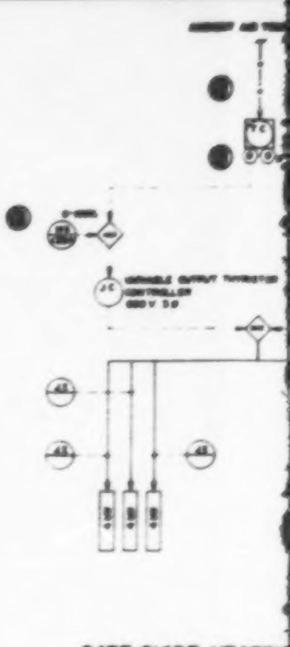
UPSTREAM RIVER LEVEL MONITORS


FLOW CALCULATED AND INDICATED
LT

DOWNSTREAM RIVER LEVEL MONITOR



AMBIENT AIR TEMPERATURE MONITORING



GATE GUIDE HEATIN

TYPICAL FOR HEATED GAT

(GATE No 3 SH

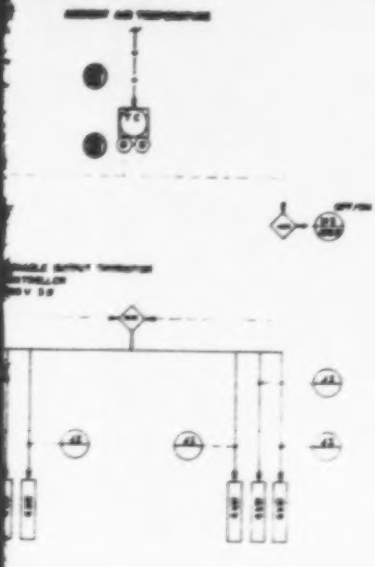
GATE SHEAVE HEATIN

TYPICAL FOR HEATED GAT

(GATE No 3 SH

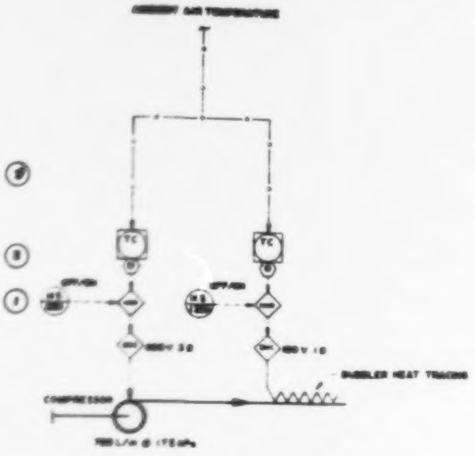
[illegible]

[illegible]


AMBIENT AIR TEMPERATURE
TC
OFF/ON
480V 3Ø

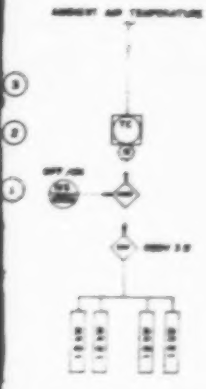
GUIDE HEATING CONTROL (a)

TYPICAL FOR HEATED GATES-No 3,4 & 5
(GATE No 3 SHOWN)


AMBIENT AIR TEMPERATURE
TC
TC
OFF/ON
OFF/ON
480V 3Ø
480V 1Ø
BUBBLER HEAT TRACING
COMPRESSOR

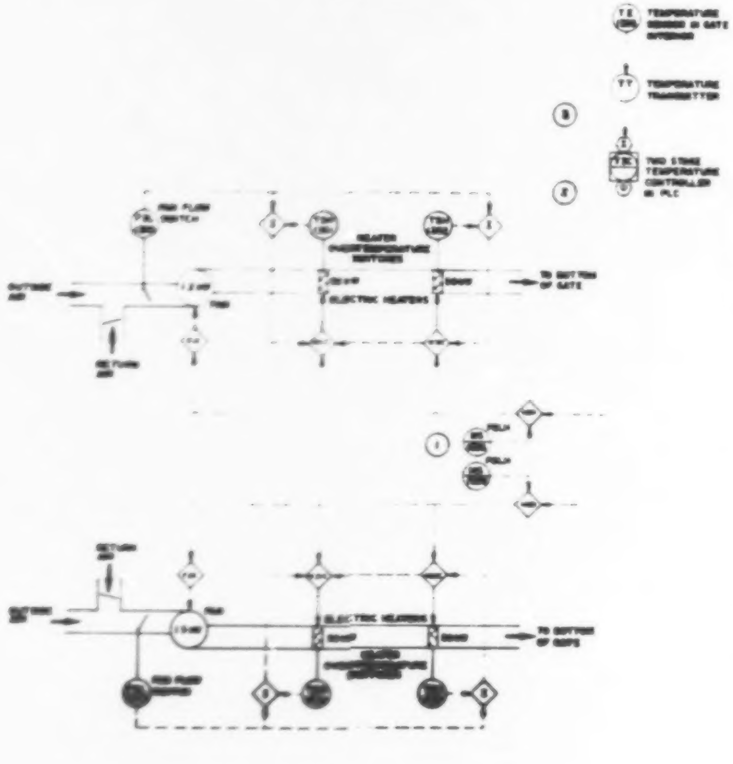
GATE BUBBLER CONTROL (c)

TYPICAL FOR HEATED GATES - No 3,4 & 5
(GATE No 3 SHOWN)


AMBIENT AIR TEMPERATURE
TC
OFF/ON
480V 3Ø

SHEAVE HEATING CONTROL (b)

TYPICAL FOR HEATED GATES - No 3,4 & 5
(GATE No 3 SHOWN)


TE
TEMPERATURE SENSOR IN GATE INTERIOR
TT
TEMPERATURE TRANSMITTER
TIC
TWO STAGE TEMPERATURE CONTROLLER IN PLC
FAN FLOW SWITCH
HEATER OVERTEMPERATURE SWITCHES
ELECTRIC HEATERS
OUTSIDE AIR
RETURN AIR
TO BOTTOM OF GATE
FOLH
ELECTRIC HEATERS
OUTSIDE AIR
RETURN AIR
TO BOTTOM OF GATE
HEATER OVERTEMPERATURE SWITCHES

INNER GATE HEATER CONTROL (d)

TYPICAL FOR HEATED GATES - No 3,4 & 5
(GATE No 3 SHOWN)

Ⓐ [illegible]

[illegible]

Ⓑ [illegible]

[illegible]

LEGEND

- INSTRUMENTATION SYMBOLOGY FOLLOWS ISA STANDARDS S5.1 & S5.3

– BOX DENOTES SHARED DISPLAY (IBM PC)

BUBBLER HEATER CONTACTOR INTERLOCK
BUBBLER STARTER CONTACTOR INTERLOCK
FI FLOW INDICATOR
FSL FLOW SWITCH LOW
FLOW COMPUTATION
FΣ FLOW SUMMATION
FAN 1 CONTACTOR INTERLOCK
FAN 2 CONTACTOR INTERLOCK
GUIDE HEATER CONTACTOR INTERLOCK
HS HANDSWITCH
HS FOLH HANDSWITCH FAN/OFF/LOW/HIGH
HS OFF/ON HANDSWITCH OFF/ON
HS 0-100% HANDSWITCH 0-100% OUTPUT
HEATING NORMAL RELAY INTERLOCK
HEATER 1-1 CONTACTOR INTERLOCK
HEATER 1-2 CONTACTOR INTERLOCK
HEATER 2-1 CONTACTOR INTERLOCK
HEATER 2-2 CONTACTOR INTERLOCK

PLC INPUT
INTERLOCK
JC POWER CONTROLLER
JI POWER INDICATOR
LI LEVEL INDICATOR
LT LEVEL TRANSMITTER
PLC OUTPUT
SHEAVE HEATER CONTACTOR INTERLOCK
TC TEMPERATURE CONTROLLER
TE TEMPERATURE ELEMENT
TI TEMPERATURE INDICATOR
TIC TEMPERATURE INDICATOR CONTROLLER
TSH TEMPERATURE SWITCH HIGH
TT TEMPERATURE TRANSMITTER

CONTROL SYSTEM
SCHEMATICS - 3

PLATE 36